

ARC WELDING

LINCOLN PRIZE PAPERS

TITLES OF PAPERS AND NAMES OF AUTHORS

"Arc-Welding—Its Fundamentals and Economics." A Treatise on the Fundamentals of Arc-welded Design and Shop Practice and an Analysis of Its Industrial Applications and World-wide Possibilities. By James W. Owens, Director of Welding, Newport News Shipbuilding and Dry Dock Company, Newport News, Va. Past Vice-president of the American Welding Society. Fellow of the American Institute of Electrical Engineers. Member of the American Society of Mechanical Engineers. Member of the Society of Naval Architects and Marine Engineers. Author of "Fundamentals of Welding—Gas, Arc, and Thermit." (*First Prize.*)

"Fundamental Principles of Arc Welding." By H. Dustin, Ingénieur, A. I. Lg. Professeur à l'Université de Bruxelles. Directeur du Laboratoire d'Essais des Matériaux. (*Second Prize.*)

"Electric Welding of Ships' Bulkheads and Similar Plated Structures." By H. E. Rossell, Commander, Construction Corps, U. S. N., U. S. Naval Academy, Annapolis, Md. (*Third Prize.*)

"Theory and Application of the Base Plate Arc-welded Rail Joint." By Frank B. Walker, Chief Engineer, Eastern Massachusetts Street Railway Company, Boston, Mass. Mem. A.S.C.E. (*Honorable Mention.*)

"Stable Arc Welding on Long-distance Pipe Lines." By B. K. Smith, President, Big Three Welding and Equipment Company, Houston, Tex. (*Honorable Mention.*)

"Arc Welding as Applied to Construction Work at the Philadelphia Navy Yard." By M. M. Kennedy and F. H. Wieland, U. S. Navy Yard, Philadelphia, Pa.

"Arc Welding of Duplicate Steel Structures," By W. H. Himes, Wilksburg, Pa.

ARC WELDING

LINCOLN PRIZE PAPERS

SUBMITTED TO

THE AMERICAN SOCIETY OF MECHANICAL
ENGINEERS

EDITED BY

EDWARD PIERCE HULSE

*Member American Society of Mechanical Engineers; American Society
for Testing Materials; American Society for Steel Treating;
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PREFACE

New information that will tend to advance the art of arc welding was the purpose and object of The American Society of Mechanical Engineers when it accepted the custody of \$17,500 given by the Lincoln Electric Company, of Cleveland, Ohio, as the

LINCOLN ARC WELDING PRIZES

to be awarded, under the rules of the competition, to those contributing the three best papers on arc welding. The aim of this competition was to encourage improvements in the art of arc welding, the pointing out of new and wide applications of the process, and the indicating of advantages and economies to be gained by its use. This fund of \$17,500 was awarded as follows: First Prize, \$10,000; Second Prize, \$5,000; Third Prize, \$2,500.

The Committee of Judges of the Lincoln Arc Welding Prize Contest included:

Mr. L. P. Alford, *Chairman*; Editor, Ronald Press Company, New York, N. Y. Mem. A.S.M.E.

Mr. S. W. Miller, Union Carbide & Carbon Research Laboratories, Inc., Long Island City, N. Y. Mem. American Welding Society.

Prof. Comfort A. Adams, Harvard Engineering School, Harvard University, Cambridge, Mass. Mem. A.I.E.E.

Mr. Henry Goldmark, Nyack, N. Y. (now New York City). Mem. A.S.C.E.

Mr. F. T. Llewellyn, New York, N. Y. Mem. A.S.C.E.

Dr. George K. Burgess, Director, Bureau of Standards, Department of Commerce, Washington, D. C.

Dr. W. F. Durand, Professor Emeritus, Stanford University, Stanford University P.O., Cal. Mem. A.S.M.E.

Prof. Robert H. Fernald, University of Pennsylvania, Philadelphia, Pa. Mem. A.S.M.E.

Advisory Members.—J. C. Lincoln, President, and J. F. Lincoln, Vice-president, Lincoln Electric Company, Cleveland, Ohio.

There were submitted seventy-seven papers. The committee completed its consideration of these papers on Apr. 7, 1928, and presented its recommendations, which were made unanimously, to the Council of The American Society of Mechanical Engineers. The awards were made at the Spring Meeting of the Society, May 14 to 17, 1928, in Pittsburgh, Pa.

The three prize papers and the two that received honorable mention, together with two other outstanding papers, contained so much of value that it has been decided to publish them for the benefit of all those interested in the advancement of and those making practical use of the art of arc welding.

THE EDITOR.

NEW YORK, N. Y.

March, 1929.

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PART I

ARC WELDING—ITS FUNDAMENTALS AND ECONOMICS

BY

JAMES W. OWENS

FOREWORD TO PART I

We have for a number of years advantageously utilized the arc- and gas-welding processes for salvage, but it was not until about two years ago, when a contract was obtained for fabricating the steel for the Mokelumne pipe line in California and the Navy Department requested bids for cruisers on which welding was extensively used, that we became interested in arc welding from a production standpoint. During this period we have extended the use of the process in the construction of the hull and machinery of ships and in the manufacture of hydraulic machinery and accessories, miscellaneous structural work, etc. As the various welding processes will undoubtedly largely supplant riveting, we urge the underwriters, classification societies, and ship owners carefully to consider the merits of arc welding as outlined in Mr. Owens' paper with a view to utilizing it more fully in future ship construction. This suggestion is also offered to executives and engineers engaged in the manufacture and design of other products in which the process can be economically applied.

H. L. FERGUSON,
President and General Manager,
Newport News Shipbuilding and
Dry Dock Company,
Newport News, Va.

MEMORANDA

As requested, I have reviewed the designs of the welded ship to be submitted by you in the Lincoln Arc Welding Prize competition and consider that the designs as indicated would make a strong and practicable ship.

H. F. NORTON,
Naval Architect,
N. N. S. & D. D. Co.

I have reviewed the designs of the welded ship and the fabrication and assembly methods to be submitted by you in the Lincoln Arc Welding competition and consider that it is practicable to construct such a ship in this yard.

E. F. HEARD,
Hull Superintendent,
N. N. S. & D. D. Co.

The proposition of building a vessel of the dimensions and type shown on your plans by the arc-welding process appears to be perfectly feasible under a system of organized training, testing, and supervision of welders such as has been developed at the Newport News shipyard. The particular design submitted is considered generally satisfactory and with some few modifications in scantlings and structural arrangements could be approved by the Bureau subject to the requirements of Sec. 26 of the rules.

D. ARNOTT,
Chief Surveyor,
American Bureau of Shipping,
New York, N. Y.

ARC WELDING

LINCOLN PRIZE PAPERS

PART I

ARC WELDING—ITS FUNDAMENTALS AND ECONOMICS

INTRODUCTION

The author purposes to show that civilization possesses in the arc-welding process a means of joining metals and of salvaging materials, with world-wide economic, commercial, military, and social possibilities of far-reaching importance. However, as the widest possible utilization of this process can be secured only by strict adherence to the scientific principles involved, he will endeavor to set forth these principles in a manner that will insure this.

In treating the subject, the definition of arc welding given in the Appendix will be adhered to, viz., "*A fusion welding process wherein the welding heat is obtained from an electric arc between the base metal and an electrode or between two electrodes.*" This definition embraces metal-arc welding, shielded metal-arc welding, carbon-arc welding, shielded carbon-arc welding, and atomic welding; but metal-arc welding being the most extensively used of the arc-welding processes at the present time, it will be understood as the process meant when the word "welding" is used.

Direct-current, bare low-carbon-steel electrodes, manual welding, and the class of material ordinarily used in the manufacture of the product under consideration—these also will be understood unless otherwise stated. In general, this material includes low- and medium-carbon-steel plates and shapes and low-carbon forgings and castings. As the conventional methods of showing welds on drawings have been used, it is suggested

that reference be made to Figs. 17 and 18 of the Appendix for a description of these methods.

Information available from contemporary sources has not been included. Unless specifically stated, all designs and specifications have been prepared and tests conducted under the author's supervision or in connection with work on which he has acted in a consulting capacity.

The author thanks the management of the Newport News Shipbuilding and Dry Dock Company, and particularly W. B. Ferguson, its production manager, for permission to include a considerable number of the company's welding and cutting standards and for the wholehearted support given him in the preparation of this paper as well as in the development of the science and art of welding.

CHAPTER I

THE ADVANTAGES OF ARC WELDING

The two principal advantages of arc welding are reduced cost ✓ and a better product. Bound up in these two are several other advantages, such as high joint efficiencies, decided savings in ✓ weight, tightness of joint, and the elimination of the noise of riveting, any one of which is often of equal and in some cases of greater importance to the user than a reduced cost of the product. To the user of pressure vessels, joints having an efficiency of 100 per cent are frequently of primary importance; to the designer of naval ships, aircraft, and other mobile structures, a saving in weight counts more than first cost; to the oil producer, tightness of joint is just as important as first cost, as it materially reduces operating costs, and to residents and property owners in a neighborhood where new steel structures are in the process of erection, the elimination of the noise of riveting tends toward the mental comfort of the residents and is an economic asset to property owners.

From the foregoing it will be seen that the many advantages of ✓ arc welding give it a unique industrial, commercial, social, and military value, which unfortunately is not as yet fully appreciated. In view of this, the most important of the advantages of the process will be considered.

Strength of Joint.—Subsequent to 1918 the ability to make ✓ consistently sound welds and the technical knowledge of welding have materially increased, yet in 1918 there were made these two pertinent statements in Captain Caldwell's report to the Emergency Fleet Corporation on the "British Admiralty's Investigations in Electric Welding":

The low stresses at which the seams open indicate that unless we can effect some improvement upon riveting we cannot utilize to the best advantage the material at our disposal and indeed suggest that we have already reached if not exceeded the stresses that riveted joints can stand with safety.

The Admiralty are quite satisfied that arc welding is at least equal if not superior to existing means of connecting two portions of the

ship's structure or for making good a defective casting, and corroboration of this view is continually being brought to notice.

In connection with the first statement, tests have shown that slippage occurs in riveted joints at 15 to 20 per cent of their

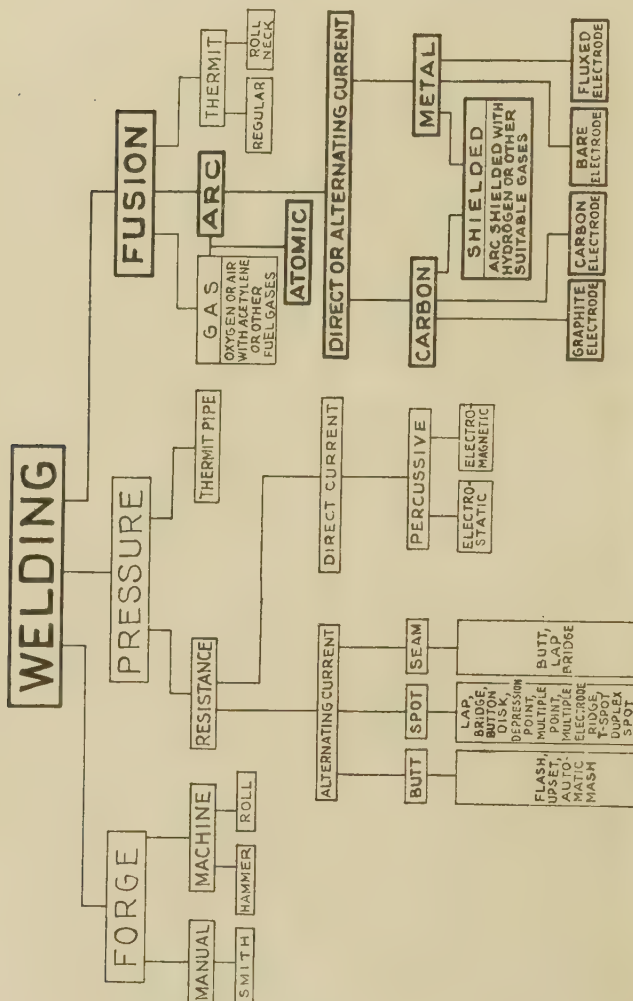


Fig. 1.—Diagram showing the arc-welding processes and their relation to other welding processes. The names, grouping, and interrelation of the various welding processes shown have been tentatively approved by the Nomenclature Committee of the American Welding Society.

ultimate strength, or less than one-half the elastic limit of the material, thus breaking the calking edge and permitting leakage to occur.



FIG. 2.—Tension test of arc-welded butt, lap, strapped butt, and tee joints in $\frac{3}{8}$ -in. ship plate.



FIG. 3.—Bending test of arc-welded butt joints in deck beams and bulkhead connections.

All members deformed as shown without failure of any weld.

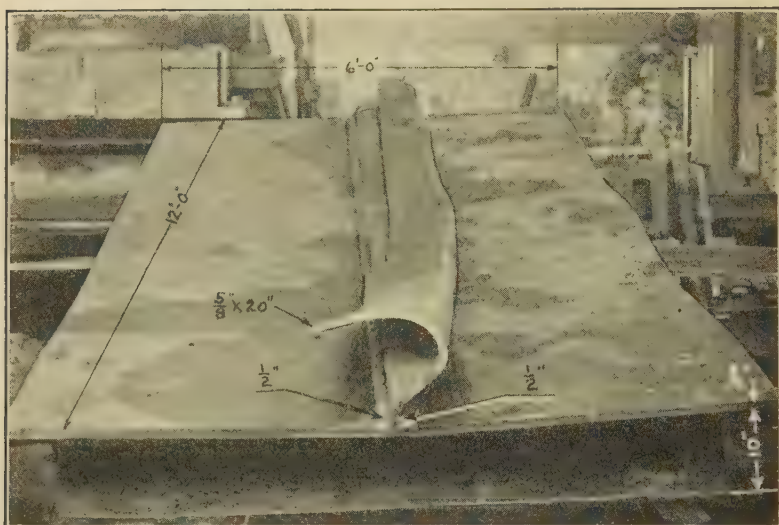


FIG. 4.—Crushing test of a ship's bilge keel.
No failure of welds.

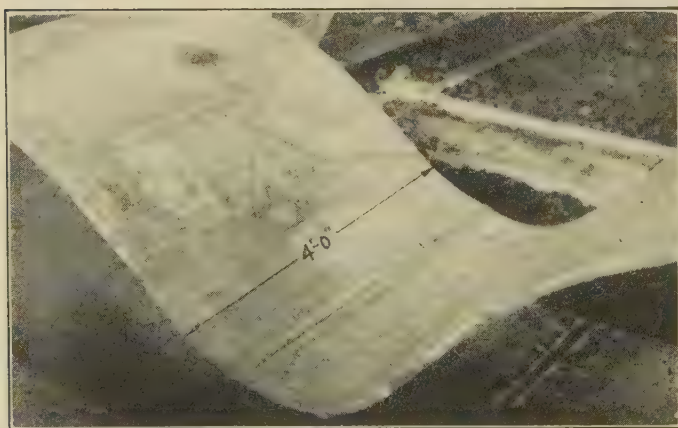


FIG. 5.—Crushing test on a 65-in. diameter pipe section welded by the Lincoln automatic carbon arc.
No failure of welds. For detail of joint see Fig. 35.

By the use of material suitable for welding, properly designed joints, and an efficient shop organization, it has been repeatedly demonstrated beyond any doubt that arc-welded joints can be consistently made with efficiencies in tension, bending, compression, shear, shock, and reversal of stress above that which can be secured by the riveted joint. The maximum



FIG. 6.—Shear and compression tests of a welded and a typical riveted pillar connection used in ship construction.

Rivets sheared. No failure of welded joint. Savings in cost of labor, material, and overhead for welded joint over riveted connection is 75 per cent.

efficiency of a riveted joint in tension, as given in the U. S. Navy Department specifications, is 89 per cent, whereas efficiencies up to 100 per cent can be obtained by arc welding, these efficiencies being based on the unwelded base metal.

The superiority of the arc-welded joint in tension, bending, compression, shear, shock, and reversal of stress is illustrated in Figs. 2 to 9, inclusive.

Savings in Weight.—As the sizes, and therefore the weight, of members used in a riveted structure are usually based on the efficiency of the joint, it follows that by the use of the arc-welded joint of higher efficiency it would be possible to reduce these

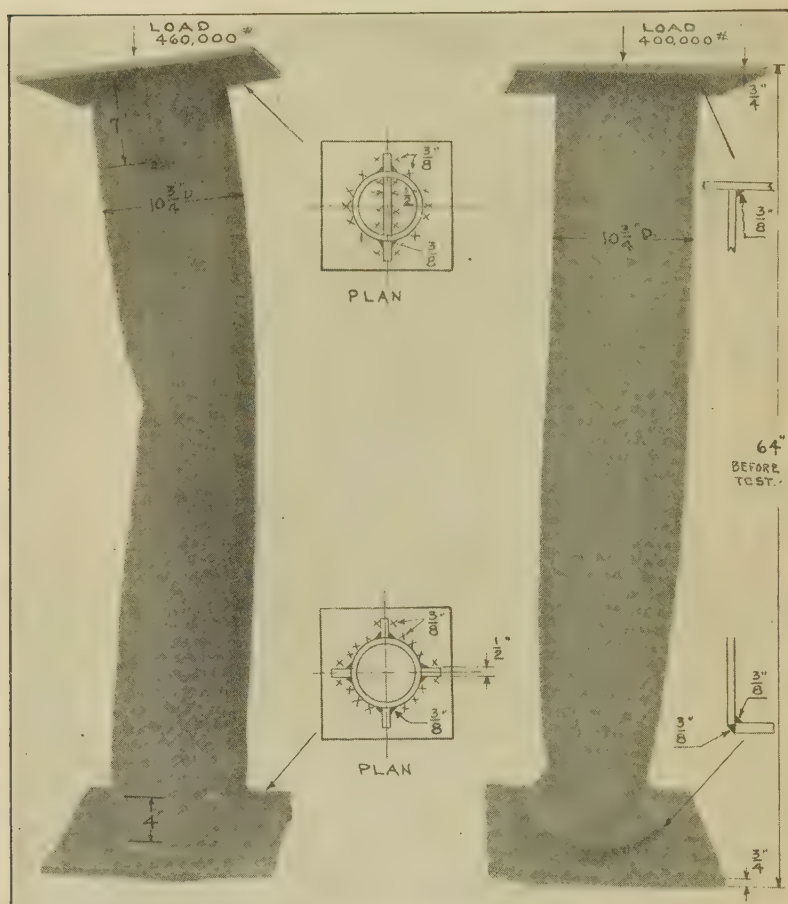


FIG. 7.—Shear and compression tests of typical welded pillar connections used in ship construction.

Pillars deformed under load as shown, without failure of welded joints.

sizes and the weight of material in the structure. Reduction in the weight of a steel structure is further made possible by the use of flat bars for stiffening members instead of angles, channels, etc.; by the elimination of connecting angles and other

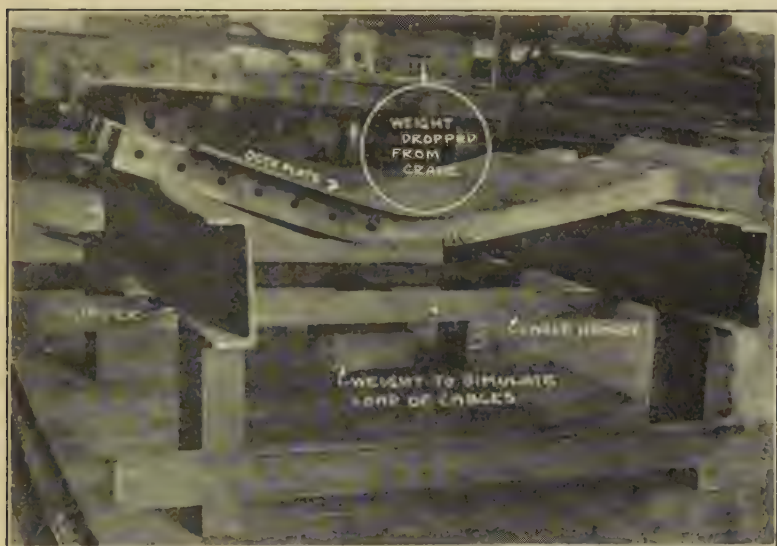


FIG. 8.—Shock test of a welded ship's deck and cable hangers.
No failure of welded joints.



FIG. 9.—An 8-in. expansion joint used on ships of the U. S. Navy.
Tested for $23\frac{1}{4}$ hr. at 150 lb. oil pressure and 4,391 alternations of stress without failure of welded joints.

flanged connections; and by the use of butt joints and narrow laps.

Finally, the mechanical limitations of riveting and calking prevent the maximum reduction in weight that it is possible to obtain with the arc-welded joint.

Savings in weight of 10, 20, and even 71.4 per cent (Fig. 56) often may be obtained by the substitution of welded steel for riveted steel, and as high a percentage by the substitution of welded steel for iron castings. Savings in weight, although much

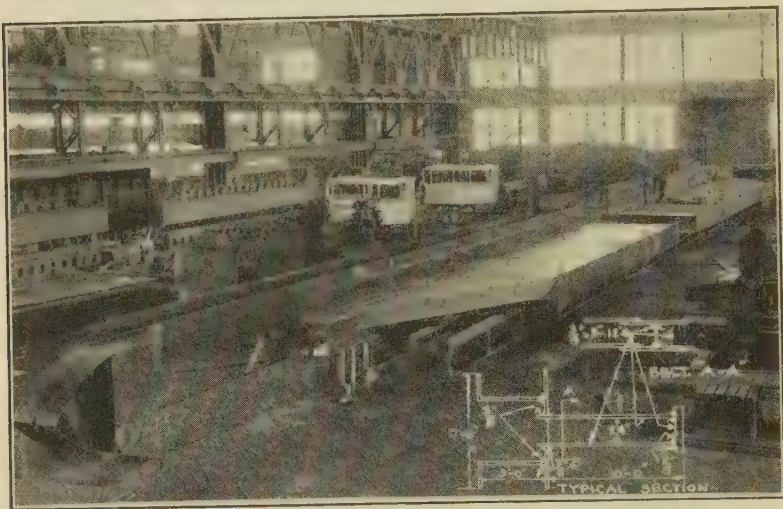


FIG. 10.—A 45-ton arc-welded battle towing target of U. S. Navy.

This type of welded target has superseded the similar riveted target. Approximately 17 have been constructed, with savings in weight of 11.9 per cent and savings in cost of direct labor of 21.2 per cent.

greater when welded steel is substituted for cast iron, are also obtained when cast steel is eliminated, for the uniformity of rolled or drawn steel permits the use of thinner material than would be considered if the article were made of cast steel. Owing to the necessity of securing the maximum reduction in weight in aircraft construction without a sacrifice of strength (which can be obtained by the use of arc and gas welding), there are very few castings used nowadays in this type of construction.

By the substitution of welded steel for cast iron, the thickness of the metal can in general be made one-fourth that required for an iron casting; however, if stiffness is of primary importance,

the welded steel should be approximately one-third that required for cast iron.

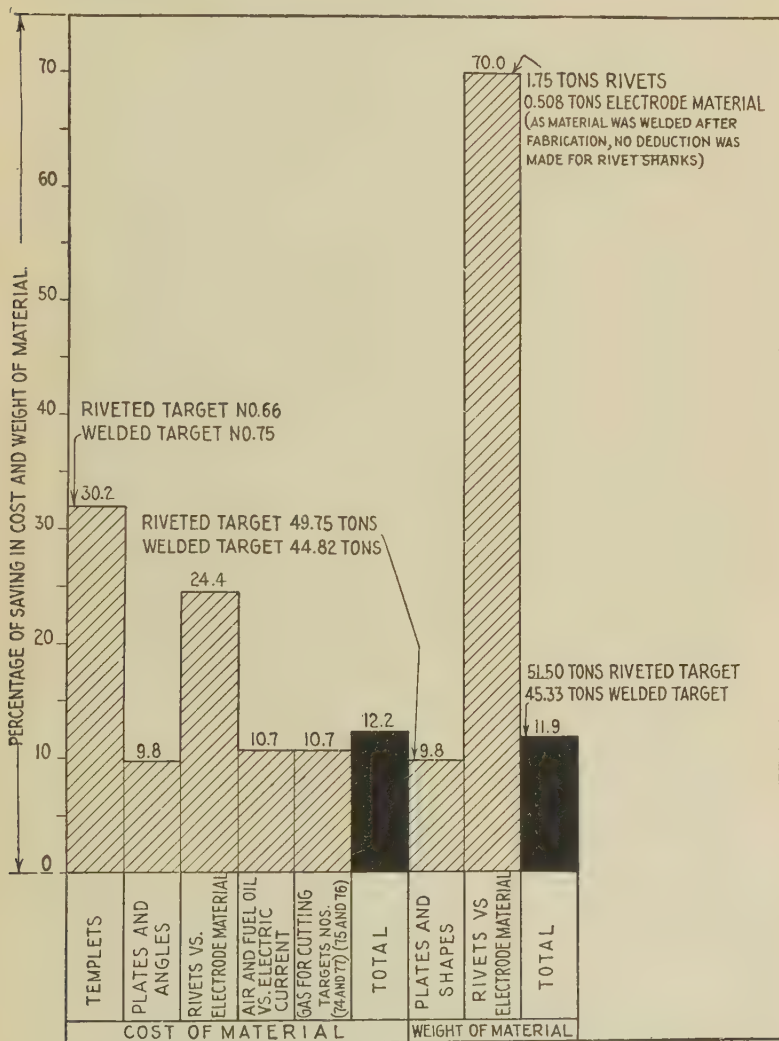


FIG. 11.—Percentage of saving in cost and weight of material.

Riveted target No. 74 versus metal-arc-welded target No. 75, unless otherwise specified.

Figure 10 shows a metal-arc-welded battle towing target under construction that is 11.9 per cent lighter than a similar riveted target. The curve (Fig. 11) is also of interest in this connection.

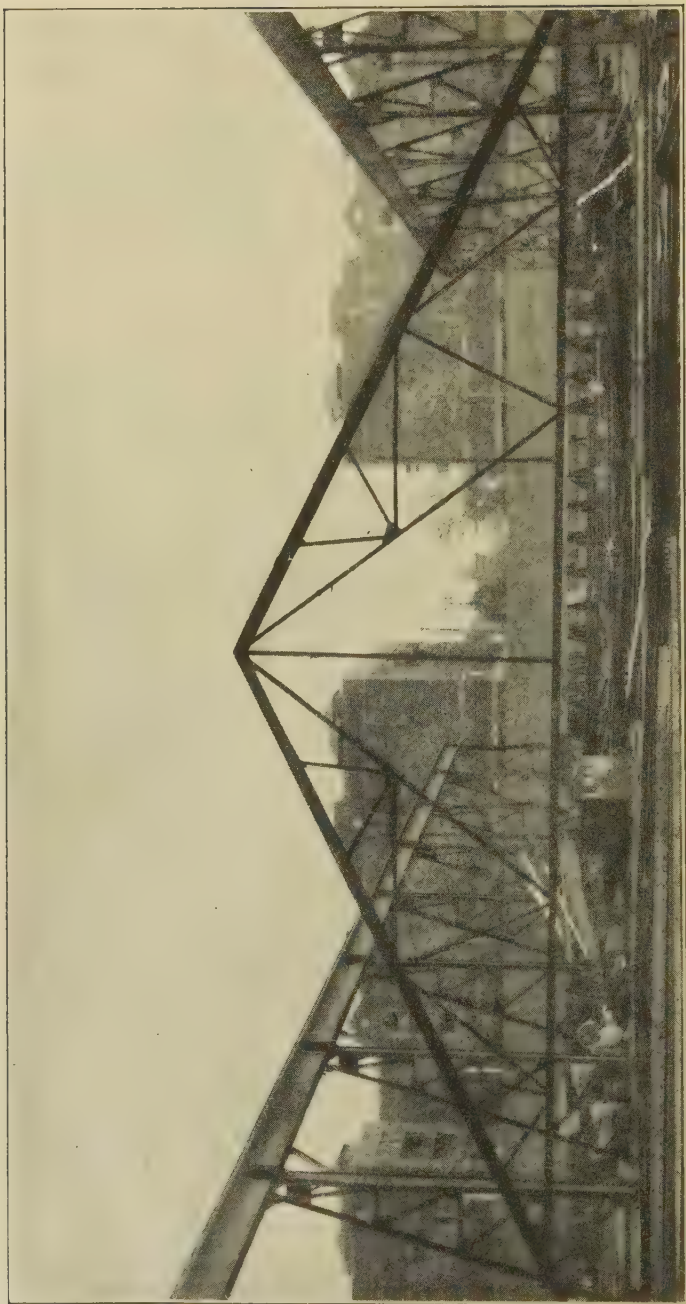


FIG. 12.—Arc-welded roof truss, 54-ft. span.

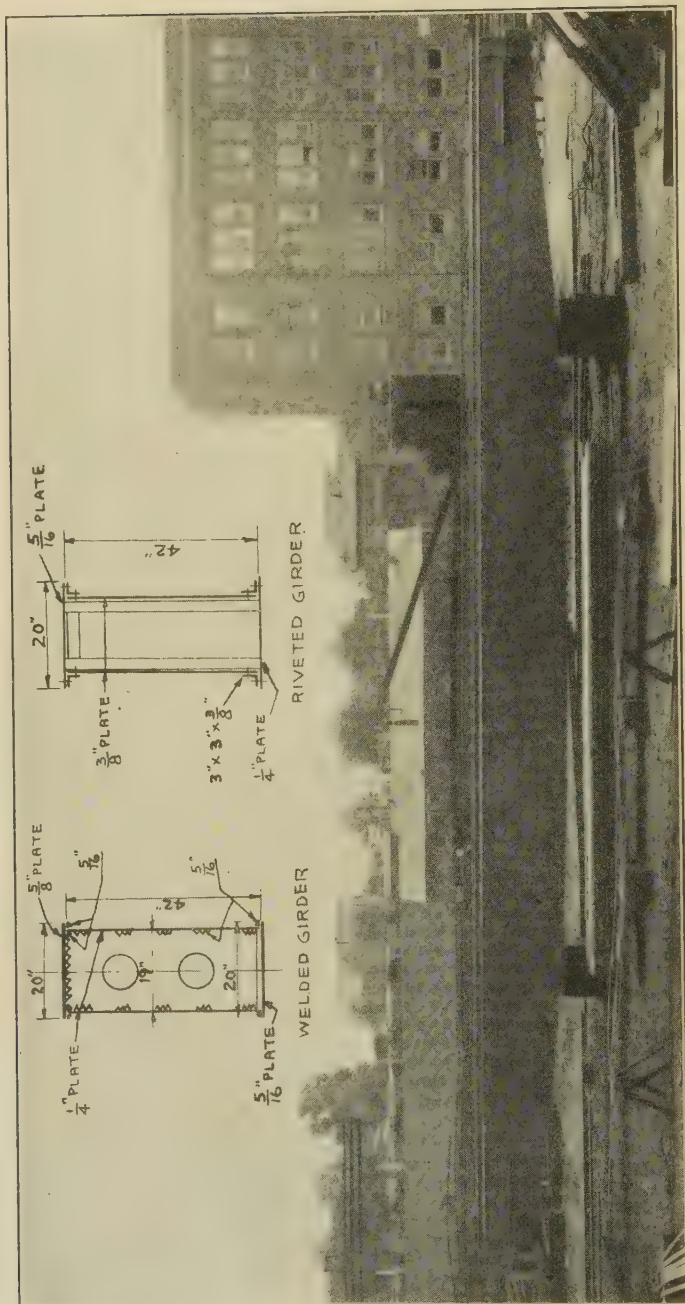


Fig. 14.—Arc-welded 10-ton 50-ft.-span crane girder for foundry cleaning room.

with similar riveted and cast-iron condenser shells that previously had been constructed, it was found that savings in

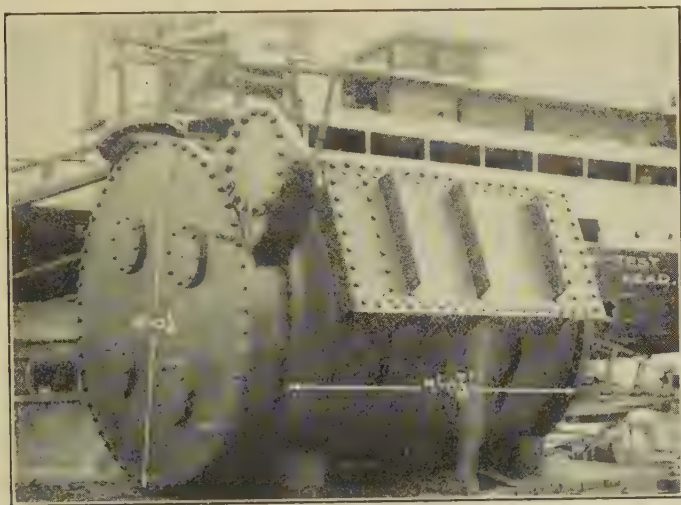


FIG. 15.—Arc-welded condenser shell for S.S. *Algonquin*.

Savings in cost of labor, material, and overhead over similar riveted shell, 9.7 per cent; and over similar cast-iron condenser shell, 22.5 per cent.

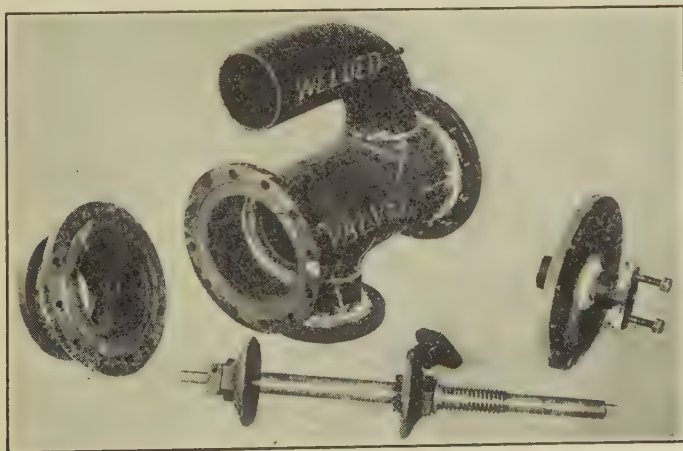


FIG. 16.—Arc-welded 4½-in. change valve tested to 300 lb. hydrostatic pressure. Savings in cost of labor and material, exclusive of patterns, 3.7 per cent.

weight of 10 and 48.6 per cent, respectively, were obtained. Figure 16 shows a completely welded valve that is 45 per cent lighter than a similar cast valve previously used.

TABLE 1.—PERCENTAGE OF SAVING IN MATERIAL AND COST OF VARIOUS PRODUCTS AND ON VARIOUS CLASSES OF WORK

Item	Class of work, product	Riveted construction		Corresponding welded construction		Percentage of saving			Remarks	Reference		
		Num- ber made	Weight each (lb.)	Num- ber made	Weight each (lb.)	Weight	Labor	Cost		Riveted	Welded	
Ship construction:												
1	Bilge keels.....	..	38,400		26,420	31.0	20.9	28.0		..	..	36B
2	Masts.....	..	27,500		22,500	18.2	15.7	15.7		..	..	48
3	Pillar connections.....	1	130	1	130	11.9	75.0	75.0	15' di. pillar used	..	..	28A
4	Battle-towing targets.....	2	103,000	2	90,670	17.5	31.0	25.6		..	..	10
5	Independent tanks.....	2	9,700	2	8,000	8.7	31.0	25.7		..	..	55A
6	Rudder.....	..	26,500		24,200	8.7	3.5	22.0		V	64	65
7	Frame bracket.....	..	339		97	71.4	43.0	50.0		..	..	56
8	Average.....	..	29,367		24,574	16.3	18.4	21.9		..	..	
Building construction:												
9	Trusses.....	..	3,077	4	2,600	15.5	50.0	34.5		..	..	I
10	Girder, crane.....	..	12,070	2	8,140	32.6	38.0	36.4		..	..	I
11	Girder, welding machine.....	..	9,550	2	8,500	11.0	47.0	24.2		..	..	II
12	Average.....	..	8,232	..	6,413	22.1	40.5	33.0		..	..	
Hydraulic:												
13	Pipe line.....	..	Tons, total	83.7	Tons, total	18.6		18.6	See Chap. IV	..	..	IV
		..	86,200	miles	72,650	18.6		18.6	See Chap. IV	..	..	36
Average for all structural work.....												
Cast construction												
Average for all structural work.....												
Item No. 13 omitted												
Castings:												
14	Condenser, shell and heads.....	1	22,400	1	11,500	48.6	-11.2	22.5	Welded condenser has c.i. heads	..	..	I
15	Gear housing.....	..	750	1	750			27.0	No attempt made to save weight	..	..	App. 57
16	Hawse pipe.....	2	9,160		6,280	31.5	15.3	42.0		V	68	V
17	Average for iron castings.....	..	10,770		6,177	42.6	-12.1	33.5		..	..	69

Actual and estimated savings in weight obtained on different types of structures and on various classes of work are given in Table 1.

Reduction in Labor.—It is possible by substituting arc-welded steel for riveted steel and castings to effect savings in labor in every phase of the manufacture of a product—design, fabrication, subassembly, and erection—provided the structure is specifically designed for welding. These savings start in the making of calculations of joint efficiencies and in the preparation

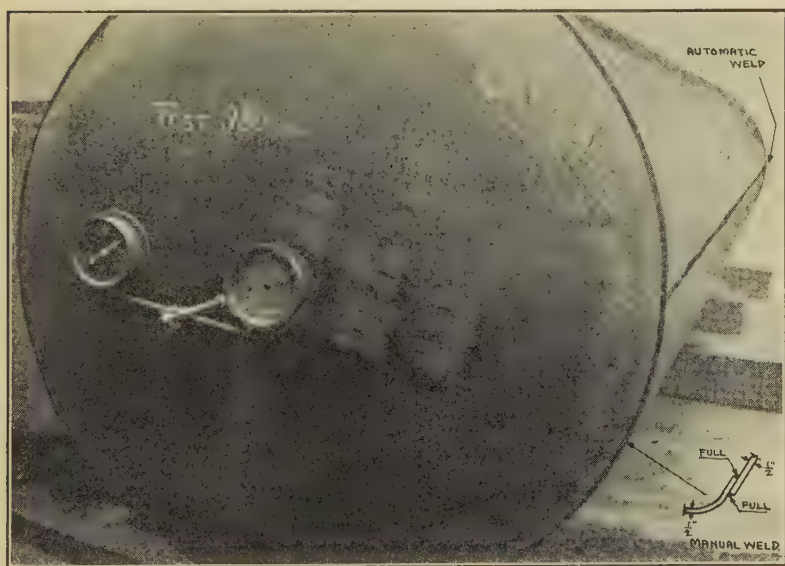


FIG. 17.—Manually and automatically metal-arc-welded tank tested by hydraulic pressure to 350 lb. and joints hammered with 10-lb. sledge.

No leaks or failure of welded joints. Longitudinal seams welded by machine shown in Fig. 27.

of drawings, for the calculations and drawings for welded structures are simpler than those of riveted construction. The elimination of the riveting schedule also saves considerable time. Shop operations can be reduced to a minimum by ordering material neat (this is usually possible when lap and tee joints are used); by using the minimum number of punched or drilled holes; by avoiding the bending and the hot-working of parts by blacksmiths and anglesmiths; and by the use of assembly jigs. The maximum amount of work should be done where handling, clamping, and jiggling facilities are available, which, of course, is



FIG. 18.—Arc-welded air receivers.

Working pressure, 200 lb. Tested by hydrostatic pressure to 400 lb. and seams hammered with 10-lb. sledge. No leaks or failure of welds.

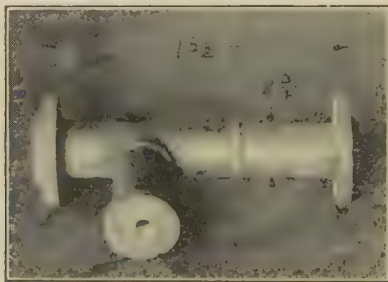


FIG. 19.—Arc-welded pipe fitting.

Tested by hydrostatic pressure to 4,560 lb. No leaks. Pipe split at this pressure.

usually in the shop; and the position of welding should be in the following order of preference, on account of the relative ease and rate of metal deposition:

Flat, Vertical, and Overhead.—In joints having flat and overhead welds it is frequently possible for the designer to increase

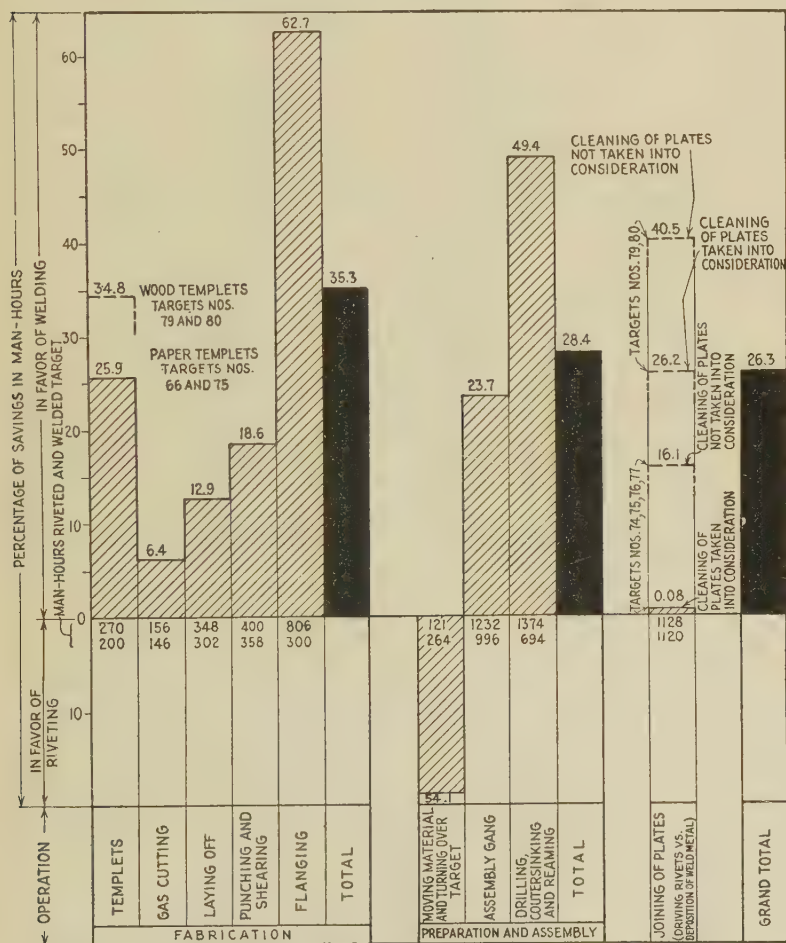


FIG. 20.—Percentage of saving in man-hours.

Riveted target No. 74 versus metal-arc-welded target No. 75, unless otherwise specified.

the size of the flat weld and decrease that of the overhead weld, so as to deposit the maximum amount of metal in the flat position. Molds and templets are very much simplified and

can frequently be made reversible—reaming is eliminated, calking is partially or wholly eliminated—and less work is required to pull the faying surfaces of joints together, a greater tolerance in assembly being permissible.

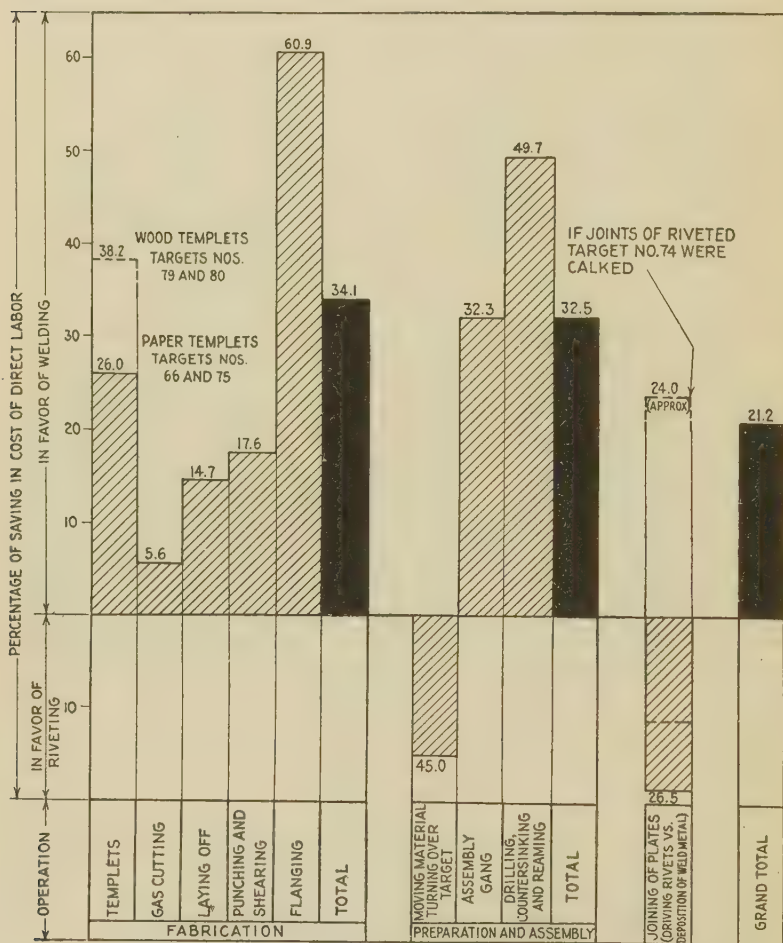


FIG. 21.—Percentage of saving in cost of direct labor.
Riveted target No. 74 versus metal-arc-welded target No. 75, unless otherwise specified.

In cases where the possibility of corrosion between faying surfaces of the joint is negligible or moisture cannot enter the faying surfaces because the joint edges are completely welded, a tolerance of $\frac{1}{16}$ to $\frac{1}{8}$ in. is permissible because friction is not taken into consideration in calculating the efficiency of a welded

joint. Figure 20 shows savings in man-hours of 26.3 per cent obtained for the various operations involved in the simultaneous construction of riveted and welded battle towing targets at the Norfolk Navy Yard.

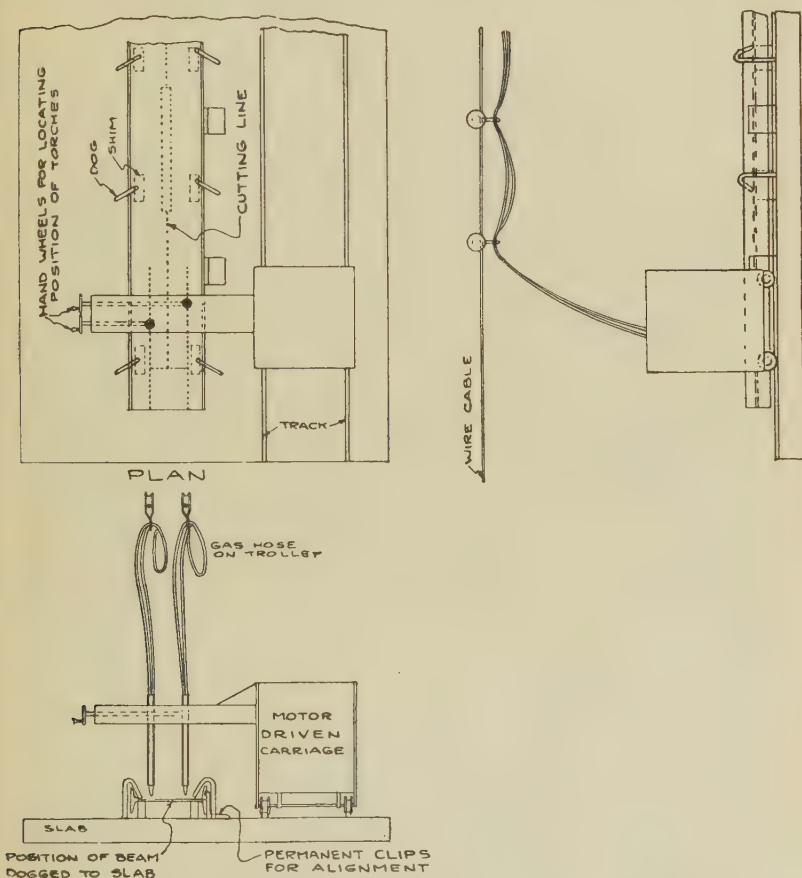


FIG. 22.—Proposed automatic gas-cutting machine for cutting ship's frames, bulkhead stiffeners, etc.

I-beam shown dogged down is being cut to form two frames, port and starboard, of welded ship shown in Chap. VII.

Savings in Time.—Less time usually is required to produce a welded-steel product than for either riveted-steel or cast products. Castings are usually the worst offenders. Exclusive of the time required to make the pattern, the normal time allowed by a foundry for making the mold for a steel casting and for baking

the mold, pouring the casting, and cleaning and annealing it is two weeks. Any one of these operations may require 24 hours or more. This time, of course, can be and is materially shortened in cases of emergency; nevertheless, the making of a steel casting requires a considerable amount of time. In repair shipyards the replacement of broken or defective castings by similar castings frequently requires that a ship remain tied up for days, with losses because of fixed charges and cessation of earning capacity amounting to thousands of dollars per day.

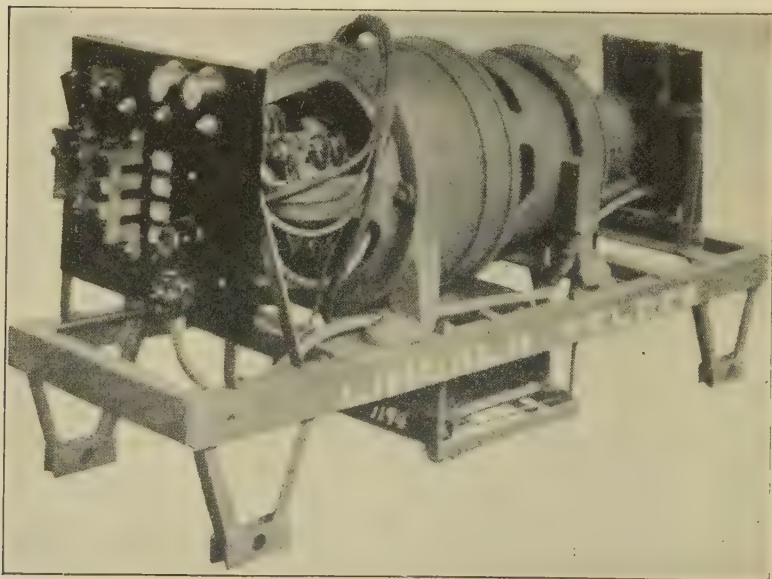


FIG. 23.—A constant-current single-operator motor-generator set.

Similar losses occur on railroads, in manufacturing plants, and with public-service companies through the failure of some cast part. In manufacturing plants, defective castings disrupt well-planned schedules in the various departments and interfere with the meeting of shipping dates. By the use of welded steel such losses and annoyances largely will be eliminated.

In general it may be stated that on structural work the savings in time are directly proportional to the savings in man-hours as considered under the heading *Savings in Labor*.

Reduction in Costs.—Not only can savings in the cost of manufacture be obtained by arc welding, but also savings in

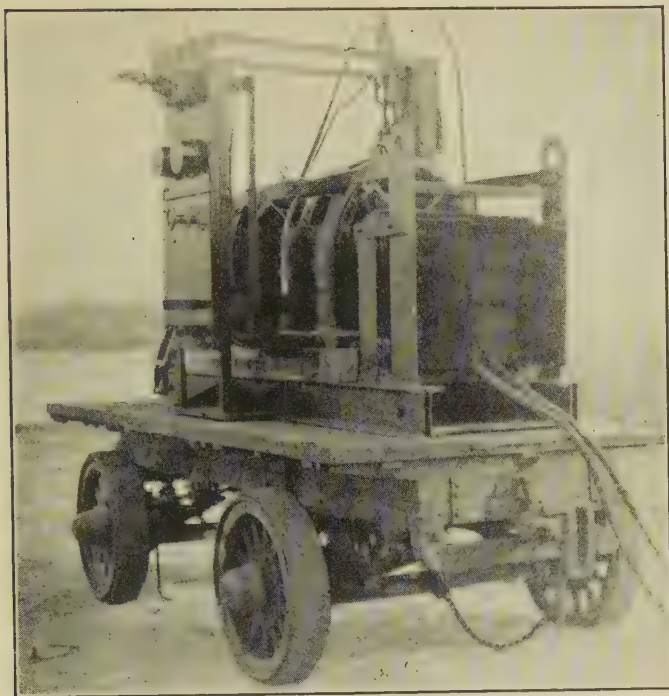


FIG. 24.—A portable constant-potential multiple-operator motor-generator set, 1,200 amp.

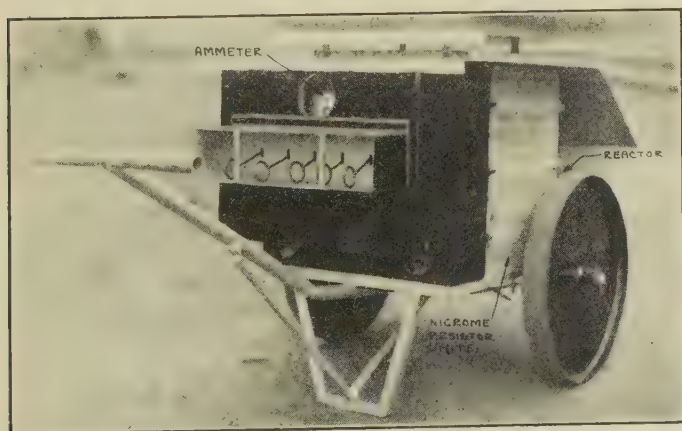


FIG. 25.—A portable nichrome resistor unit provided with reactor and ammeter.

upkeep and in operation. All of the various advantages of the arc-welding process that have been considered have a bearing upon each of these groups.

Lower Cost of Manufacture.—Considering the cost of manufacture first, we find that reduction in labor and weight of material used are the two principal items. Again, welding makes possible less transportation and handling. Further reductions in welding costs can be obtained by a detailed attention to shop technic, *e.g.*, the use of 18 instead of 14-in. elec-

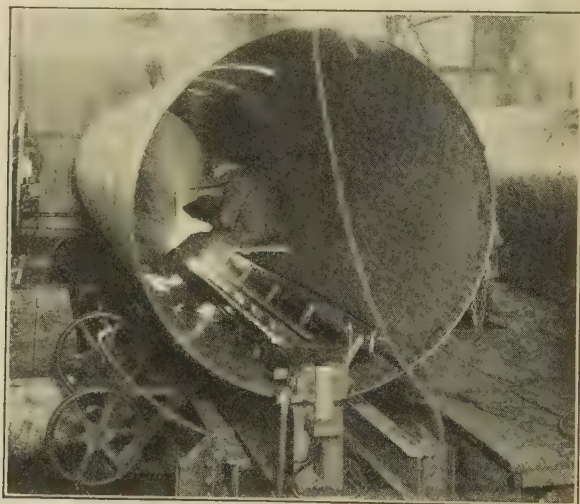


FIG. 26.—Automatic carbon-arc-welding machine for pipes.

This machine was used for welding the longitudinal seams of the 90-mile pipe line referred to in Chap. IV.

trodes; the use of $\frac{3}{16}$ and $\frac{1}{4}$ -in. electrodes instead of $\frac{1}{8}$ and $\frac{5}{32}$ -in. electrodes; the employment of the maximum current values suitable for the work; the use of automatic welding machines, of which three different types are shown in Figs. 26, 27, and 28; and the weld gage shown in the Appendix. The weld gage insures the deposition of the minimum amount of metal consistent with strength. All of these means of lowering the cost of manufacture hinge upon a drafting-room organization that is thoroughly trained to design welded structures and upon an efficient welding department.

Bound up in this manufacturing cost is that of overhead, which in foundries, blacksmith, and anglesmith shops will in general

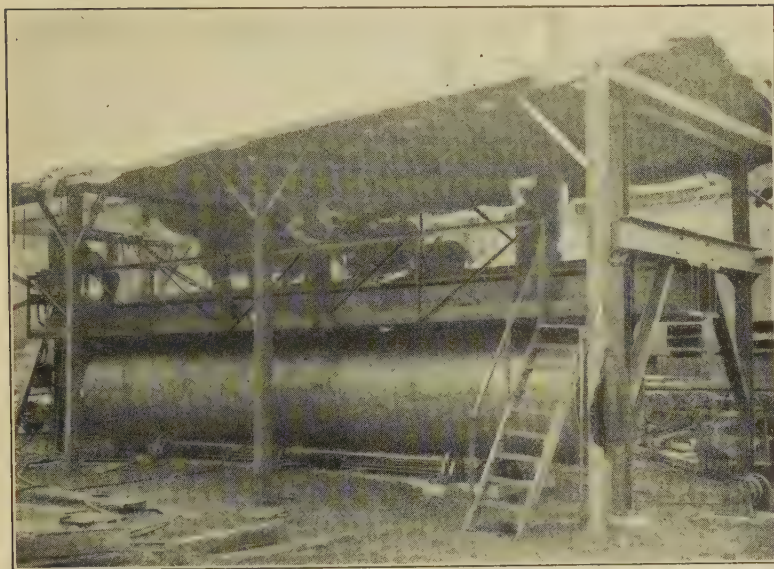


FIG. 27.—Automatic metal-arc-welding machine for pipes.

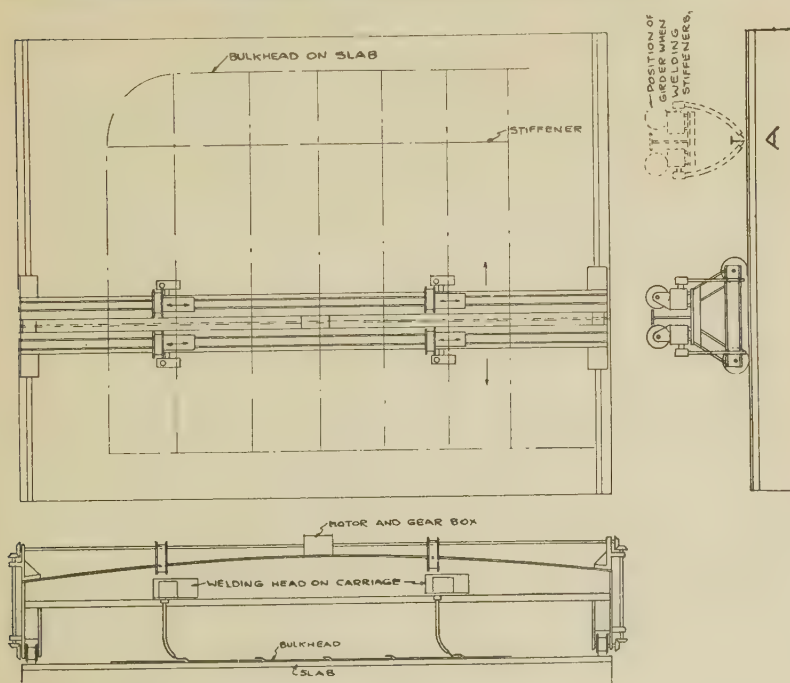


FIG. 28.—Proposed automatic metal-arc-welding machine for welding bulkheads, girders, etc.

be higher than that of plate and welding shops. By designing to keep work out of shops having a high overhead, there will be a proportionate lowering of overhead expense, and therefore the cost of manufacture. As welding makes possible a reduction in the total man-hours required in the various trades, there is also a corresponding reduction of overhead. A reduction in the stock of patterns and spare parts and a reduction in the capital invested in equipment are also features to be considered in the reduction of overhead.

Lower Cost of Upkeep.—The elimination of leaky connections is one of the principal items involved in a lower upkeep cost, which for ships is especially high. In the oil fields it has been possible by the use of welded joints to eliminate the force formerly required to inspect the mechanical joints.

Lower Cost of Operation.—Any leakage in containers and pipe lines decreases the efficiency of the plant or installation. Similarly, by the saving of weight in mobile structures, less power is required, and in ships a greater deadweight capacity is obtained, all of which tend toward a reduction in the cost of operation.

CHAPTER II

BRIEF OUTLINE OF THE FUNDAMENTALS OF DESIGN AND SHOP PRACTICE NECESSARY TO INSURE THE EXTENSIVE ECONOMIC USE OF ARC WELDING

DESIGN

Composite Design.

Composite Joints.—As a general policy, a composite joint (that is, one in which both welding and riveting are used to obtain strength) should be avoided, because the cost of welding is often merely added to the cost of the riveting; and the presence of the rivet holes tends to cause leaks and to decrease the effective section of the joint when only one edge of a lap joint is welded. On the other hand, by the use of a full, continuous fillet weld on one edge of a single-riveted lap joint, it is possible to secure a joint efficiency of over 75 per cent of that of a double-riveted joint, and by the use of two full, continuous fillet welds, a 100 per cent efficiency. Composite joints would also be economical and desirable for parts of a ship requiring a large number of assembly bolts to pull together the faying surfaces of a joint. Tests have shown that there need be no fear of rivet slippage of such a joint as the rivets and weld function together; on the other hand, the use of a light continuous weld to calk a leaky riveted joint, the failure of which would endanger life, should be prohibited. A light calking weld may, however, be used under certain conditions to calk the joints in new pressure vessels, provided the joint has first been made tight by mechanical calking and danger of rivet slippage removed by subjecting the vessel to the usual hydraulic tests. ✓

No paint should be used between the faying surfaces of a composite lap joint having two continuous fillet welds. When only one fillet weld is employed and water can enter the faying surfaces, iron-oxide paint should be used instead of red lead as the fumes from the iron-oxide paint are not harmful to the welder and the porosity of the weld is less. (See paper by the author in the April, 1927, issue of the *Journal of the American* ✓)

Welding Society.) The rivets should preferably be driven on completion of welding to avoid leaks developing at the rivets. The width of the lap should be that of a standard single-riveted lap.

Composite Structures.—Where wholly riveted and completely welded joints are employed in the same structure, the design should be such that there is no likelihood of the slippage of the riveted joints in service requiring the composite and/or the welded joints to take a greater proportion of the load than they were designed originally to stand, with the possibility of their failure.

Welding Galvanized Material.¹—If galvanizing is required, it should be done after welding. When necessary to weld galvanized material, remove the zinc coating on and adjacent to the welding surfaces by grinding, by sand blasting, or with sulphuric acid. All traces of sulphuric acid should be removed with lime water.

Welding Light Material to Heavy Material.—Careful consideration should be given by a designer to the welding of light material to heavy material because of the differences in thermal capacity. A current value that is suitable for the heavy material would probably be too high for the light material, and *vice versa*. If too high for the light material, unsatisfactory workmanship would result because of holes being melted through the metal, unless a backing strip is used. The welding of light material (for example, $\frac{1}{8}$ in. thick) to heavy material is entirely satisfactory if spot-welded or if a fillet weld is made on the edge of the thinner material. Should a fillet weld be desired on the edge of the heavier part, the lighter metal should preferably be not less than $\frac{1}{4}$ in. in thickness.

The Influence of Carbon on the Physical Characteristics of Base and Filler Metals.

Base Metal.—Steels low in carbon, manganese, phosphorus, sulphur, and silicon are, in general, satisfactory for welding. Those having a high carbon content are brittle in the fusion zone, which seriously affects the ability of the joint to withstand bending and fatigue stresses. Again, when high-carbon steels are welded, the high resistance of the joint to the stresses set

¹ While the procedure outlined is in general correct, the author has, subsequent to the presentation of the paper, attached galvanized stiffeners to galvanized floor plates of a ship by intermittent welds and without the removal of the galvanizing in way of the weld.

up by the cooling of the weld and base metals tends to cause failure of the weld during welding or prior to the application of the working stresses.

Ship plate, which has a carbon content of approximately 0.25 per cent, is satisfactory for welding, but particular care must be taken to prevent failure from residual stresses, when butt joints are used, by peening, properly reinforcing and/or strapping the weld. High-carbon and special steels should not be welded when used as strength members of a ship; there would, however, be no objection to the welding of small local clips on these members if spaced at wide intervals. In some cases the use of these steels can be avoided when welding is substituted for riveting, owing to the high efficiency of the welded joint in low and medium steels. Steel having a carbon content not exceeding 0.15 per cent is in general preferable, and it is hoped that as welding is more generally used, consideration will be given to a more extensive utilization of steel having approximately the following analysis, *viz.*, that called for in specification A-78-27, grade B, of the American Society for Testing Materials:

C	Mn	Ph	S
Not over 0.20 for $\frac{3}{4}$ in. plates and under, and not over 0.22 for above $\frac{3}{4}$ in.	0.35 to 0.60	Not over 0.06	Not over 0.05
Yield point, pounds per square inch (minimum)	Ultimate strength, pounds per square inch	Elongation, percentage in 8 inches (minimum)	
0.5 tensile strength, but not less than 27,000	50,000	25	

The following data were secured from inspection reports on 10,000 tons of this material, $\frac{3}{8}$ and $\frac{1}{2}$ in. thick:

C	Mn	Ph	S
0.13	0.40 to 0.46	0.018 to 0.020	0.031 to 0.045
Yield point, pounds per square inch	Ultimate strength, pounds per square inch	Elongation, percentage in 8 inches	Reduction in area, percentage
32,800 to 37,700	53,640 to 61,480	27 to 31	52 to 59

Filler Metal.—The principal components of metal electrodes and welding rods are iron, carbon, manganese, nickel, silicon, sulphur, phosphorus, aluminum, copper, and zinc.

Carbon appears in electrodes and welding rods in about the same proportion as it does in steel. Ingot-iron electrodes and welding rods should have no carbon, but frequently show one one-hundredth of 1 per cent, which is negligible from the practical welding standpoint and is the purest commercial iron obtainable.

Low-carbon-steel electrodes as covered by the specifications of the American Welding Society permit of two carbon limits, *viz.*, 0.06 (within which limit are included ingot-iron electrodes) and 0.13 to 0.18.

Electrodes of both of the foregoing chemical analyses are used for the welding of mild-steel structural shapes, plates, low-carbon-steel forgings, castings, and cast iron. An analysis of a typical deposit made with a bare low-carbon-steel electrode will show the following percentages of the various elements: carbon, 0.05; manganese, 0.11; phosphorus, 0.02; silicon, 0.07; and sulphur, 0.011.

In order to secure a higher carbon content in the deposit, it is found necessary for the carbon content of the electrodes to be 0.90 to 1.10 per cent. Electrodes of this composition, however, are used only for the welding of high-carbon steels and worn surfaces where great resistance to abrasive wear is desired and where peening and/or machining is not necessary.

It is recommended that the filler metal for arc welding comply with the specifications of the American Welding Society, *Bulletin* 2, issued Apr. 1, 1920, and revised May 1, 1921.

Types of Joint (Appendix, Figs. 1 to 4).—All joints when welded restrain the cooling weld metal, causing residual stresses to be set up in the joint or parts of the structure remote from the welded joint. The amount of this restraint, and therefore the value of the residual stress set up, is very largely dependent on the rigidity of the joint and/or structure caused by the carbon content of the base metal, its design, and welding procedure. Joints in low- and medium-carbon steels the design of which is such that there is very little restraint to the cooling metal have a low residual stress when welding is complete and the joint has cooled, while those in high-carbon steels and of relatively great fixity will on cooling have a very high residual stress. Failure from this cause will occur when the cooling stress exceeds the

tensile strength of the weld or of a section of the structure that is weaker than the weld.

In view of the foregoing, all joints can be classified in four principal types depending on their ability to relieve heating and cooling stresses. These types of joints in their relative order of restraint to the heated and cooling base and weld metals are the *non-rigid*, *semi-rigid*, *rigid*, and *fixed*. ✓

In the *non-rigid* type the cooling stresses are transverse to the joint; therefore, the resistance to the contracting forces of the weld and base metals is low and uniformly distributed throughout the joint. These stresses are primarily relieved by a deflection of parts and secondarily by a pulling together of the faying surfaces. Such welded joints will never fail from their own residual stresses, and if failure does occur it will always be caused by some external force; for example, another weld with high cooling stresses in an integral part of the structure. ✓ Typical joints of this type are the lap, tee, and corner. In general this type of joint should be used in preference to the other types when metal-arc welded. A butt joint should be used for the longitudinal seams of tanks and containers subject to breathing stresses. ✓

The *semi-rigid*, *rigid*, and *fixed* types of joints are butt joints in which the cooling stresses are in the same plane as the joint. In the *semi-rigid* type the weld is made from one side only, or from the open side of the V if of single-V form; the members are angularly placed prior to welding, and plates are free to move and approach each other as welding proceeds. The cooling stresses are primarily relieved by a free movement of parts and secondarily by elongation and compression of the base and weld metals. The weld is made in one layer, which is started at one end of the joint and completed at the other end. When completed, the joint edges should be parallel the entire length of the joint. This joint is particularly suitable for gas and manual carbon-arc welding, but is unsuitable for structural and for ship work in general on account of the required movement of the plates during welding, which introduces problems in assembly and erection. The angularity of the joint edges of steel plates should be approximately $2\frac{1}{2}$ per cent when gas or carbon-arc welded and $1\frac{1}{2}$ per cent for metal-arc welding. Non-ferrous metals require a slightly greater angle of the joint edges. On completion of welding there is a high-tension stress in a section ✓

about 3 in. long at the ends of the weld in this type of joint and a compression stress in the central section. The ends should therefore be reinforced by weld metal or a strap, unless reinforced by the structure into which incorporated.

The *rigid* type of joint, like the semi-rigid type, permits free movement of parts during welding, but the joint edges to be welded are placed parallel prior to welding and held in place by tack welds spaced equidistant on approximately 6-in. centers for the length of the joint. The cooling stresses are primarily relieved by a movement and/or deflection of parts, which are partially restrained by the tack welds and secondarily by elongation and compression of the base and weld metals. The stresses in this joint are alternately tension and compression, and although negligible in low-carbon material of $\frac{1}{4}$ in. and below, they might cause failure in heavier material and even in light material if the carbon content is high. The welds of such joints should be reinforced on at least one side and preferably on both sides of the weld, in order to provide a section of weld metal having a greater strength than that of the base metal, or by reinforcing the weld on one side and providing a strap on the other side.

The *fixed* type of joint affords no movement of parts except by preheating or mechanical means; for example, the use of a jack. The cooling stresses are therefore wholly relieved by elongation and compression of the base and weld metals. An example of this type of joint is a welded patch in the porthole of a ship the joint of which is of single-V or double-V form. When this joint is used, careful consideration should be given to provide proper reinforcing and/or strapping as described. A welded strap is desirable when this type of joint is used in important strength members and is required in some specifications.

In the rigid and fixed types it has been found that when it is necessary to make the weld in more than one layer the weld metal in the first layer frequently fails because the sectional area through its throat is too small to resist the cooling stresses induced by the larger mass of base metal to which it is fused. There are several methods of preventing or reducing such failures to a minimum, one or more of which may be employed. These methods are: keeping the base metal cool on each side of the weld, by conducting the heat away with water-cooled copper strips or other means; welding slowly with comparatively small

electrodes, so as to reduce the heat in the base metal to a minimum; and peening each bead or layer, so as to compress the metal. This last method is the most important and effective of the three methods given.

Forms of Joint (Appendix, Figs. 8 to 12).—There are five fundamental forms of joint; namely, the *edge*, *butt*, *lap*, *corner*, and *tee*; and while there are many modifications of these, owing to the shape of sections used, the shape of the joint edges, and the distance they are spaced apart, nevertheless all can be related back to these fundamental forms.

No attempt will be made to describe or illustrate all of these variations. The most commonly used, however, are shown in the Appendix, and their descriptive names are given.

Beveling of Joint Edges (Appendix, Fig. 22).—The joint edges of single-V and double-V butt joints are beveled to a total angle of 60° for metal-arc welding. A 90° bevel is, however, necessary when heavy cast-iron sections are welded by the metal arc and when steel studs are employed, and in a few other cases; for example, Detail *B*. It is usually not necessary to bevel the edges of butt joints when the material is $\frac{7}{32}$ in. and under. Material over $\frac{7}{32}$ in. and up to $\frac{1}{2}$ in. is not beveled for automatic carbon-arc welding and sometimes not for automatic metal-arc welding. For tee joints in $\frac{1}{2}$ -in. material and below it is also usually not desirable to bevel the edges of the abutting section; however, beveling from one or both sides is sometimes advantageous and even necessary in heavier material. ||

Width of Lap (Appendix, Tables 3 and 4).—When composite joints are employed, the minimum width of lap should always be that of a single-riveted lap (Table 3), so as to provide a proper bearing value for the rivets. In completely welded joints the width of the lap may be reduced to the thickness of the thinner plate joined, if the maximum saving in weight is desired and no assembly bolts are used. On the other hand, the minimum allowable width is sometimes covered by specifications, and special permission would have to be obtained to use laps narrower than those specified. The Navy Department requires that the minimum lap should be twice the thickness of the thinner material welded; the American Bureau of Shipping, twice the thickness of the thinner material welded, plus 1 in.; and Lloyds Register of Shipping that of a single-riveted lap (See Table 4-A and B). Table 4-C gives the minimum widths of lap when

assembly bolts are used in the faying surfaces and the joint is completely welded. These widths have been obtained by a series of tests by the author and are the minimum which is necessary to prevent failure at the edge of the plate when punched and assembled.

Thickness and Width of Strap (Appendix, Table 4).—The thickness of a strap used to reinforce a welded butt joint should be one-half the thickness of the thinner section joined. In the case of double-strapped closed-butt joints the thickness of each strap should be the thickness of the thinner section joined. The width of all straps should be governed by the conditions shown in the sketches (Table 4).

Opening between Joint Edges.—The opening between the joint edges of double-lap and double-parallel-lap joints (Appendix, Figs. 11-B and 11-D) should be not less than five times the thickness of the thinner material welded owing to the combined thermal effect of the fillet welds on the base metal between them and the high bending moment in this section, if the opening is less than that specified (see also Appendix, Table 4).

The author has conclusively proved this by test specimens and service tests on the battle towing targets of the Navy. Corner, single-bevel-tee, double-bevel-tee, single-V-butt, and double-V-butt joints should be provided with a $\frac{1}{8}$ - to $\frac{3}{16}$ -in. opening between joint edges. The faying surfaces of lap and straight tee joints should be designed to be in intimate contact; however, a $\frac{1}{16}$ -in. opening would be satisfactory. If the opening is $\frac{1}{8}$ in. or greater, the size of the fillet weld should be proportionately increased so as to obtain the necessary fusion area required for the weld. The opening may be measured with the weld gage (Appendix, Fig. 21).

Location of Work and Position of Weld.—It is usually preferable to do work in the shop rather than in the field; that is, if much welding is involved. Again, it is usually not necessary to specify whether a weld should be made in the shop or in the field; however, in cases where a weld is specifically required to be made in the field the symbol indicating this (Appendix, Fig. 17, Note 2) should be used. Good overhead metal-arc welds can, however, be made, but welding in this position costs about 30 per cent more than flat welding and should be avoided whenever possible. The designer should bear this in mind, and the shop should consider turning the work so as to permit welding in the most favorable positions.

Surface Finish and Reinforcement of the Weld.—The surface finish of all welds should be uniform and free from deep indentations or depressions in the weld-metal and fusion zones. Slight indentations are not harmful and even aid the inspector to determine whether good penetration has been obtained. The weld metal should in single-V-butt welds come entirely through the bottom of the V as a uniform bead. This can be insured by spacing a backing strip of copper or steel approximately $\frac{1}{8}$ in. away from the bottom of the V or by having the copper strip in contact and providing it with a slight groove. The backing strip is removed when welding is complete.

It is recommended that a gage similar to that shown in the Appendix be used on strength welds to insure uniformity of the deposit. Butt welds with both surfaces flush should never be used except by special permission. When butt welds are used, they should in general be reinforced on both sides and the weld metal should overlap the base metal on each side of the weld. The depth of each reinforcement and the amount of overlap should be approximately one-fourth the thickness of the base metal (Appendix, Fig. 23). Butt welds in pipe lines which cannot be reinforced on the inside of the pipe may be reinforced on the outside only. In general it is not necessary to reinforce fillet welds, reinforcement being required only when a weld can be made on only one edge of a lap or tee joint and when a standard fillet does not give the required strength or when reinforcement is necessary to take care of corrosion.

Strength Welds.—Arc-welded joints can be made stronger than the material which they join, but as it is not always necessary to do so, the designer should figure the strength of each joint used, so as to obtain the required efficiency, up to 100 per cent, at the lowest cost. In lap and tee joints any degree of joint efficiency may be obtained by employing light, full, or reinforced fillet welds and by making these welds continuous and/or intermittent as requirements indicate (Appendix, Fig. 24). A continuous fillet always should be made on the side requiring tightness, whereas on the other side a light continuous or an intermittent weld frequently will suffice. Butt welds always should be designed to have the maximum efficiency.

The size of all welds in strength joints is based on the thickness of the thinner material joined and their strength on the thickness of the throat of the weld metal; that is, the shortest distance

through the weld. The size of a fillet weld is expressed by the length in inches of its shortest leg (Appendix, Fig. 15). Fillet welds in strength joints should in general be $\frac{3}{8}$ in. standard. It is usually not found necessary to use standard fillet welds smaller than $\frac{3}{16}$ in. nor larger than $\frac{1}{2}$ in. When intermittent fillet welds are used exclusively for the attachment of a member, the ends of the member should be welded on both sides. In butt welds the location and size of each reinforcement should be given and each reinforcement specified in terms of its depth and width (Appendix, Figs. 18 and 23).

Calk Welds.—Fillet welds when used for the calking of riveted joints may be as small as $\frac{3}{16}$ in., provided the joints have not been subjected previously to working stresses. When joints with loose rivets are welded for tightness, strength welds should be employed. A reinforced weld should be used for the calking of riveted butt joints to avoid possible failure from residual stresses.

Working Stresses and Design Values for Welds.—Welded joints subject to bending and/or alternating stresses should be designed so that deflection will take place in the base metal and not in the weld. This is possible by reinforcing or by reinforcing and strapping the weld. Both edges of lap and tee joints should be welded when strength is essential. A continuous or an intermittent strength weld may be used on both edges or a continuous weld on one edge and an intermittent weld on the other edge, to prevent a bending stress in either weld. In general use 36,000 lb. per square inch for the ultimate tension shear and compression strengths of the weld metal and 9,000 lb. per square inch for its working strength. The efficiency of light, full, and reinforced fillet welds may be expressed in percentages and their ultimate strength in pounds per linear inch, as given in Tables 5, 6, and 7 of the Appendix. The elastic limit of the weld metal can be taken as 32,000 lb. per square inch. These values for ultimate strength are very conservative, as will be shown in Chap. III.

SHOP PRACTICE

The Gas Cutting Torch.—The full utilization of the gas cutting torch and the arc weld would practically do away with other methods now used for the fabrication and construction of steel structures. In so far as the cutting torch is concerned, the

present cost of cutting and punching holes as compared with available mechanical methods largely restricts its use to such members or parts of a structure which owing to their shape or size cannot be economically sheared, punched, or otherwise mechanically fabricated.

The fewer number of holes required in completely welded structures and the different functions performed by these holes make the cutting torch more economical for the punching of holes and the cutting of members in both shop and field. Again, as the most efficient sections for welded design are not now rolled, and as it is possible to obtain such sections by splitting I-beams, channels, and H-sections, the cutting torch also can be effectively utilized to secure such sections; particularly is this true if the torch can be operated mechanically. In view of this, various types of automatic gas cutting machines should be obtained or developed for various classes of work.

Figure 22 is a sketch of an automatic gas cutting machine which it is proposed to develop to fabricate the frames and bulkhead stiffeners used in the design of the arc-welded freight ship described in Chap. V. If desired, this machine can be so developed that several torches can be used simultaneously on the section and cuts made and holes punched from patterns provided by the mold loft.

Welding Machines.

Single Operator.—The variable voltage or constant energy motor-generator set is one of the most efficient methods of obtaining direct current when a limited number of welders are employed, when such welders are distributed over a wide territory, and for automatic machines (carbon and metal-arc) on repeat production work. Such machines are of the single-operator type, although two independent generators frequently are driven from one motor. They can be obtained in various sizes up to 500 amp.; however, 200- to 300-amp. sets are in more general use. Figure 23 is a machine of this type.

Multiple Operator.—In plants employing over 30 welders, where most of the work requires the concentration of four or more welders, and where such grouping of welders is distributed over a wide area, as in shipyards, it has been found that 1,000-, 1,250-, or 1,350-amp., multiple-operator, 60-volt, constant-potential motor-generator sets afford the most flexible welding

source that can be obtained commercially, and upkeep costs are very much less than that of single-operator sets. A group of multiple-operator machines may be installed and connected in parallel in a sub-station, or individual machines may be mounted on a rigid bed-plate and provided with crane lifting hooks for convenient handling (Fig. 24). Individual resistors for multiple-operator machines should have a current range from 15 to 300 amp. and preferably should be built up of nichrome units, so as to obtain relatively unvarying current values. These units should be mounted in a framework on a light truck together with a 300-amp. reactor and an ammeter (Fig. 25). It is quite feasible to mount several constant-current generators on the same shaft and drive them by one or more motors, and for some classes of work this arrangement will be preferable to the multiple-operator set.

Automatic Welding Heads and Travel Mechanisms.—There are a limited number of commercial automatic carbon- and metal-arc-welding heads obtainable, each having certain welding and operating advantages. In using these heads it is necessary to move either the work or the head to secure linear fusion, and although mechanisms for this purpose are procurable from the manufacturers of the welding heads, it frequently is necessary for the user to develop mechanisms suitable to his particular work.

Figure 28¹ is a sketch of a proposed automatic metal-arc-welding machine for the welding of bulkheads, girders, etc., for the welded freight ship described in Chap. V. This machine consists of a girder having a span of 60 ft. and a clearance above the slab of 4 ft., thus permitting it to span the maximum depth of bulkhead and clear the deepest girder that it would be required to weld. The girder of this machine will be provided with a variable-speed motor for moving it in the direction of the transverse seams of the bulkheads and welding them by the automatic heads that are mounted on one or more carriages, as shown. Each of these carriages also is provided with a variable-speed motor to secure travel in the direction of the bulkhead stiffeners and for the welding of them to the bulkhead plating.

¹ Subsequent to the writing of this paper the Newport News Shipbuilding and Dry Dock Company has constructed two 38-ft. machines of this type. A skid constructed of angle-iron framework is used instead of the welded slab suggested.

Welding Slabs (Figs. 29 and 30).—What generally is known as a “bending slab” is assuming a new and important function, that of rigidly holding work while it is being welded, so as to prevent warping. In the welding shop this slab is therefore known as the “welding slab,” and no welding shop is equipped properly unless it has one or more permanently located slabs. Welding slabs should be of cast iron, built up of units about 5 ft. square and mounted on concrete piers, and have a working face that is perfectly level. Welding shops also should have a number of portable slabs of various sizes with the working face at right angles to the main slab, so as quickly to assemble tanks and structural work without the use of assembly holes. The welding shop of one of the large manufacturing companies has a welding slab entirely covering its floor and with several portable right-angle slabs.

To bolt down effectively the large plate areas to be manually or automatically welded (as ship’s bulkheads) presents a problem which it is proposed to solve by one of the following methods:

(a) Providing a welding slab of large area, with ample headroom, and with holes of 4-in. diameter spaced on 15 in. centers. The work to be dogged down is provided with 1-in. nuts, tack-welded to it to correspond with these holes, wedge bolts being screwed into the nuts from the under side of the slab. The nuts are knocked off on completion of welding. This method has the advantage that there are no holes to weld up. This is the type of slab that it is proposed to use for the automatic welding machine shown in Fig. 28 and the bulkheads described in Chap. V.

(b) Punch $1\frac{5}{16}$ -in. holes in the bulkhead plating for bolting to the slab described and welding up same on completion of the job.

(c) Constructing a slab of magnets, with faces of the pole pieces forming its work surface. To avoid magnetic interference of the arc, it is proposed to provide “cut-out switches” for each magnet, so that when a weld is being made within a magnetic field, the interfering magnet can be cut out. The advantage of such a slab is obvious; however, its first cost would be very high.

Jigs and Fixtures.—All properly equipped welding shops should be provided with universal jigs and fixtures for the class of work undertaken. For pipe work, standard metal patterns should be used so as accurately and quickly to cut bevels and openings for branches for various sizes of pipes.

Preparation of Joint Surfaces.—All defective parts should be cut out, exposing only sound metal for welding. Joint surfaces must have all scale, rust, and grease thoroughly removed prior to welding, and a wire brush should be employed to keep the surfaces clean during welding.

Methods of Assembly (Figs. 51 and 52).—In general, assembly bolts of $\frac{1}{2}$, $\frac{5}{8}$, or $\frac{3}{4}$ in. diameters and spaced 15 to 24 or even 36 in. apart will be found the most practical method. When used, the wedge type of bolt should be considered. On the other hand, assembly offers a wide field for ingenuity and is an item on which considerable savings can be effected. Many jobs can be assembled by tacking alone, thereby eliminating the cost of welding the assembly holes on completion of the job.

Among other means of assembling is the use of angle clips or nuts that are tacked to one of the members and bolts are employed to pull the members together, thus avoiding the use of assembly holes or reducing their number to a minimum. These clips or nuts can be knocked off easily on completion of welding.

When lap and tee joints are used, it is possible completely to assemble the structure prior to welding. This is desirable as it reduces warping to a minimum and production is increased by avoiding delays incident to assembly. On tank work and repeat production jobs, the use of jigs should be considered.

Provisions for Taking Care of Expansion and Contraction.—When butt joints are welded, the weld or the base metal will fail owing to residual stresses, if expansion and contraction are not taken care of properly. The principal methods employed for taking care of these stresses, in addition to using suitable types of joints previously mentioned, are: thoroughly peening each bead or layer with a pneumatic tool; rounding the corners of patches; dishing the patch; corrugating the sheets adjacent to the weld, and heating the corrugation ahead of the weld; using wedges or jacks to keep the joint edges apart, when the joint is not restrained to any considerable extent; preheating the joint, when of some appreciable size; and finally thermal separation of the joint edges by the heating of a parallel restraining member while the joint itself is being preheated.

Preheating and Annealing.—One of the principal reasons for preheating is to insure good fusion. When an arc weld is to be made in non-ferrous materials, the job should be preheated.

It is also advantageous to preheat and slowly cool cast-iron sections, if the work is of a character that permits. Slow cooling reduces hardness in the fusion zone. Annealing always should be considered for repairs to defective castings, welds in high-carbon steels and for low and medium steels on some classes of work.

Sequence of Deposition.—The order or sequence of deposition of the weld metal is not important in lap, corner, or tee joints, because heating and cooling stresses are readily relieved; however, the sequence of deposition is sometimes important when

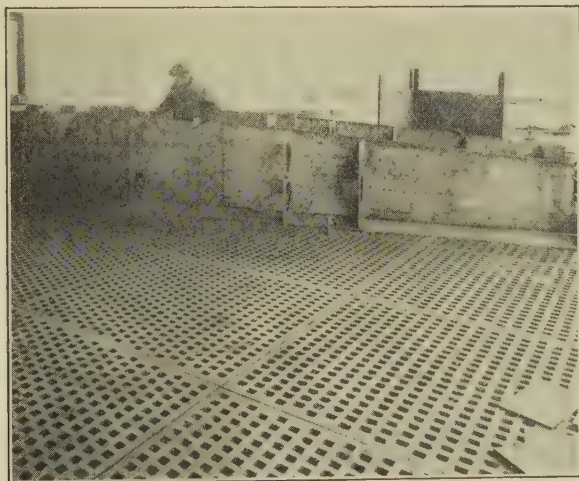


FIG. 29.—Welding slab, showing crane girder (Fig. 14) dogged down for welding.

butt joints with some appreciable degree of fixity are welded. In general, two sequences are found desirable for manual welding, the first being the *step back* or the method whereby the weld is made in short sections, about 3 in. long, the welding of each section being started about 3 in. ahead of the previous section and the metal deposited toward this section. The second method is the *alternate* and is similar to the step back except that the sections are deposited as remotely as possible from each other until the joint is completely welded. Butt welds should be made prior to fillet welds, in cases where the completion of the fillet weld tends to increase the fixity of the butt joint, and for this reason the second fillet of a strap should not be made until the butt weld is completed.

Warping and Its Prevention.—One or more of the following methods may be used to prevent or reduce warping to a minimum: use metal-arc welding instead of carbon-arc welding; peen each bead or layer; jig or dog down the work thoroughly (Fig. 29) or employ additional stiffening members; keep work cool by water or air; do not remove dogs or take work out of jig until cool; use fillet welds in preference to butt welds; reduce the size of fillet

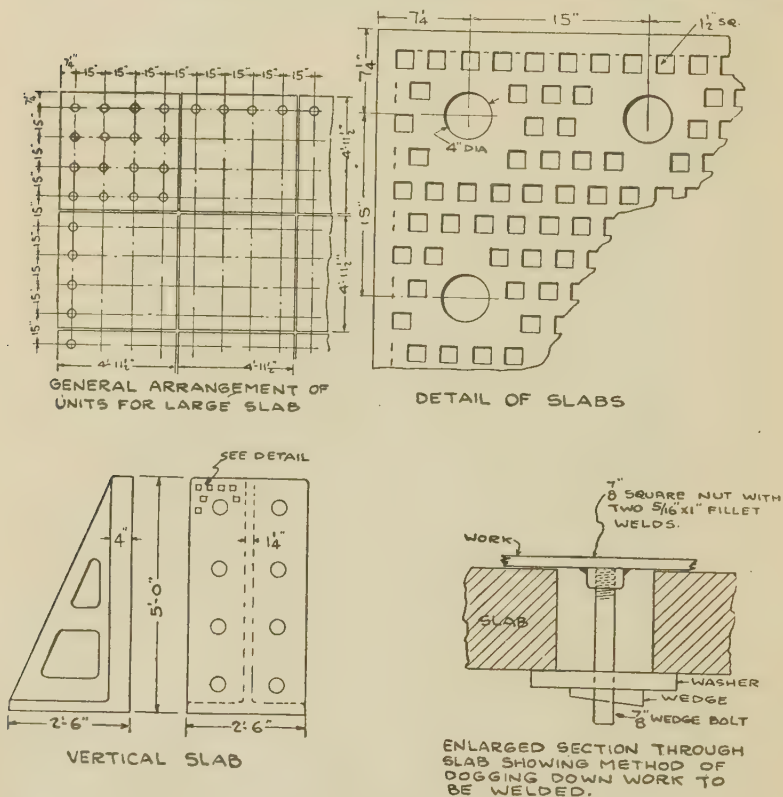


FIG. 30.—Arrangement and detail of welding slabs.

welds to a minimum; use intermittent welds instead of continuous welds; and keep the thermal input low by the use of the smallest electrodes and lowest current values with which it is possible to secure good fusion.

Sizes of Electrodes and Welding Rods.—Metal electrodes of $\frac{1}{4}$ and $\frac{3}{8}$ in. diameters have been satisfactorily used by the author at the Norfolk Navy Yard for the manual welding of

lap and tee joints in $\frac{3}{8}$ - to 1-in. plating of battle towing targets. However, $\frac{1}{8}$ -, $\frac{5}{32}$ -, and $\frac{3}{16}$ -in. electrodes and welding rods are the sizes that most generally are found useful. The majority of ship and structural work can be satisfactorily welded with $\frac{3}{16}$ -in. electrodes. The usual sizes of carbon electrodes are $\frac{1}{4}$, $\frac{5}{16}$, $\frac{3}{8}$, and $\frac{1}{2}$ in.

Current Values for Electrodes.—No fixed current values can be specified for the various sizes of metal electrodes owing to the number of variables involved; the highest current possible, however, should in general be used for the thickness of material, form of weld, and position in which it is welded (see Table 2).

TABLE 2.—CURRENT VALUES AND SIZES OF BARE ELECTRODES FOR VARIOUS THICKNESSES OF BASE METAL

Thickness of base metal, inches	Size of electrode, inches	Amperes
$\frac{1}{16}$	$\frac{3}{32}$	80-125
	$\frac{1}{8}$	125-160
$\frac{1}{8}$	$\frac{1}{8}$	125-160
	$\frac{5}{32}$	160-180
$\frac{3}{16}$	$\frac{5}{32}$	160-180
	$\frac{3}{16}$	180-200
$\frac{1}{4}$	$\frac{5}{32}$	160-180
	$\frac{3}{16}$	180-225
$\frac{3}{8}$	$\frac{5}{32}$	160-180
	$\frac{3}{16}$	180-225
$\frac{1}{2}$	$\frac{5}{32}$	160-180
	$\frac{3}{16}$	180-225
$\frac{5}{8}$	$\frac{5}{32}$	160-180
	$\frac{3}{16}$	180-225
	$\frac{1}{4}$	200-260
$\frac{3}{4}$	$\frac{5}{32}$	160-180
	$\frac{3}{16}$	180-225
	$\frac{1}{4}$	200-260
1	$\frac{5}{32}$	160-180
	$\frac{3}{16}$	180-225
	$\frac{1}{4}$	200-260

The Welding of Cast Iron.—The use of the metal arc for the repair of fractured cast iron does not require the preheating of the part to be welded; consequently the removal of the broken part is not necessary. The principal disadvantages in this

method are that the use of steel studs is required at the joint surfaces to prevent cooling stresses pulling the weld metal away from the cast iron in the fusion zone and that it is difficult to make the weld tight. Provided good fusion has been secured and no cracks have developed during welding, porosity can be

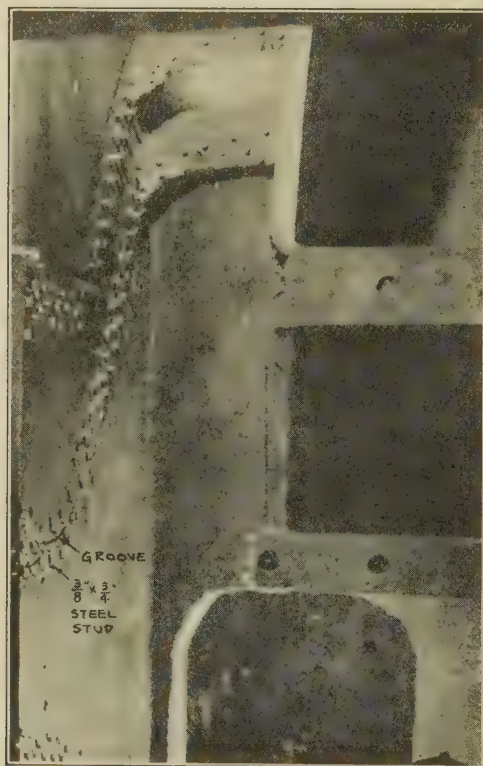


FIG. 31.—Cast-iron high-pressure cylinder of S.S. *Rondo* showing studding preparatory to welding.

remedied by painting the welded joint prior to test with a sal-ammoniac paste on the test side. Success depends on careful studding and slow welding. Studs should be $\frac{3}{8}$ in. in diameter, $\frac{3}{4}$ in. long, and spaced approximately $1\frac{1}{4}$ in. apart. The successful repair at the Norfolk Navy Yard in 1918 of the high-pressure cylinder of the 11,000-ton S. S. "Rondo," shown in Figs. 31 and 32, and the welding of a 10-ft. crack in the head of a main condenser on the battleship "West Virginia" should give confidence in this method of repair when properly executed.

The chief engineer of the "Rondo" on Feb. 24, 1919, wrote to the Norfolk Navy Yard as follows:

High-pressure valve chamber was examined after a run of 5,850 miles. The welding and patches put on at the Norfolk Navy Yard show no signs of working and are in good condition at the present time.

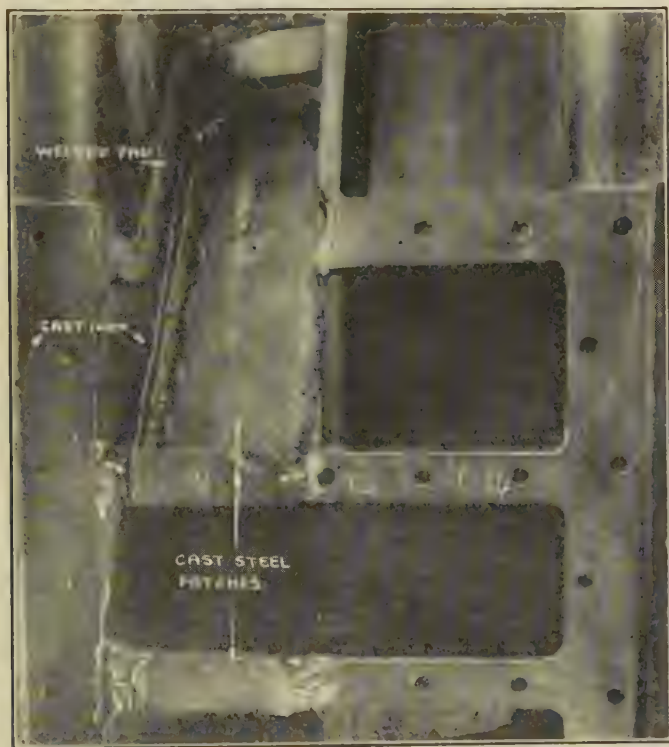


FIG. 32.—Cast-iron high-pressure cylinder of S.S. *Rondo* showing two ports welded.

The Welding of Non-ferrous Metals and Alloy Steels.—The welding of non-ferrous metals and alloy steels can be accomplished satisfactorily by the atomic hydrogen and shielded arc-welding processes. Carbon- and metal-arc welding also can be satisfactorily employed in many instances; the work, however, usually requires preheating and a reversal of polarity. Some manufacturers specialize in electrodes and welding rods for these materials, and it is recommended that they be consulted when important work is undertaken.

CHAPTER III

A DISCUSSION OF THE PRINCIPAL STOCK OBJECTIONS TO THE USE OF ARC WELDING

The author is aware that this paper may call forth some objections to the utilization of arc welding on structural work, where the failure of a joint might endanger human life and involve property damage. An attempt therefore will be made to give an answer to those objections most commonly encountered; *viz.*, the human element in welding and inability to test the finished weld; weakness of welded joints under fatigue and shock; low ductility of weld metal and lack of homogeneity in the welded joint; fixity of a welded joint prevents adjustment under load; lack of service tests; porosity of the weld; corrosion of the weld; arc welding not suitable where electric power supply is not available; difficulty of repair; and that welding costs more than riveting.

The Human Element in Welding and Inability to Test the Finished Weld.—As in riveting, the successful application of arc welding hinges not only upon good workmanship but on suitable equipment (manual and automatic) and materials, on proper design, and on good technical supervision. Until recently not enough thought was given to any of these, with the result that failures occurred that have prejudiced persons who had not sufficient information to analyze them. Failures from similar causes also have occurred in riveted construction, and still occur although less frequently. Means similar to those that have been and that are being taken daily to prevent failures of riveted structures are therefore now being taken for welded construction. Some of these means are:

(a) The appointment by the American Welding Society, of Structural Steel, Pressure Vessel, Procedure Control, and other technical committees; the appointment of technical committees by the American Society of Mechanical Engineers, the American Institute of Electrical Engineers, and other technical bodies, to investigate certain phases of welding; and the publication of training courses for welders by the American Welding Society.

(b) The preparation of welding specifications by the various regulating bodies.

(c) The establishment of technical welding courses in the universities and of practical welding courses by trade schools and industrial concerns.

(d) The employment by industrial concerns of trained engineers to design the various welded structures and competent supervisors to direct and control welding technic in the shop, thus taking it out of the hands of the practical welder.

(e) Requiring welders, assigned to strength work, to demonstrate their ability to make equally sound welds in the flat, vertical, and overhead positions. This requirement is being adhered to in the various navy yards and has been taken care of adequately in the Navy Department's specifications for new construction by requiring that the average ultimate tensile strength of all welders' test specimens made in the flat, vertical, and overhead positions shall be 52,000 lb. per square inch and that the minimum strength of no specimen shall be less than 45,000 lb. per square inch.

Fifty-one metal-arc welders who recently were tested at the Newport News Shipbuilding and Dry Dock Company, to comply with the specifications of the Navy, obtained a minimum tensile strength of 45,776 lb. per square inch, a maximum of 71,648, and an average for all positions of 60,922. A single-V butt joint in $\frac{3}{8}$ -in. ship plate, direct current, and bare low-carbon-steel electrodes were used. *It should be noted that the fifty-one welders who were tested composed the entire arc-welding force, that apprentices were included, and that not one welder failed to pass the test.* The author's experience has shown that welders who pass such a test will make consistently sound welds, if furnished with suitable materials and equipment. For complete test results see Table 3 and Fig. 33.

(f) The extensive use of automatic welding machines.

In some types of automatic welding machines the human element is entirely eliminated and welds can be made that are consistently sound.

(g) Providing good welding equipment and suitable materials.

(h) Providing each operator's equipment with an ammeter, so that the supervisor can see that his welders are using the current values that he previously has instructed them to employ, such values being the highest suitable for the work, so as to insure good penetration (see Table 2).

(i) Using conservative ultimate design values for arc-deposited metal.

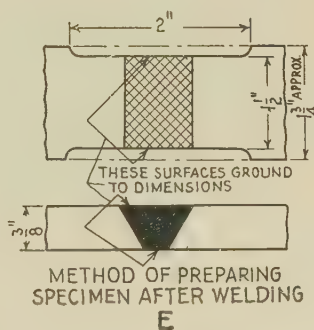
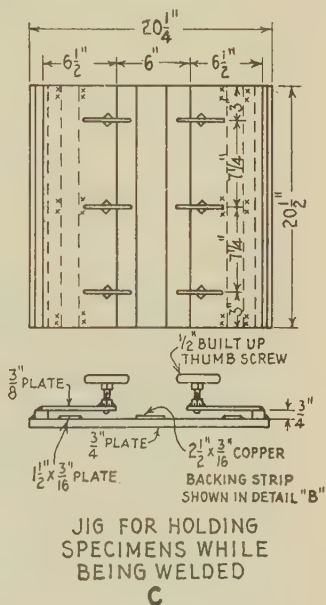
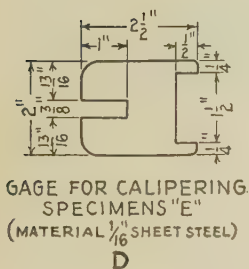
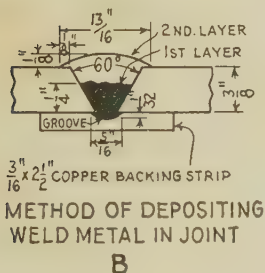
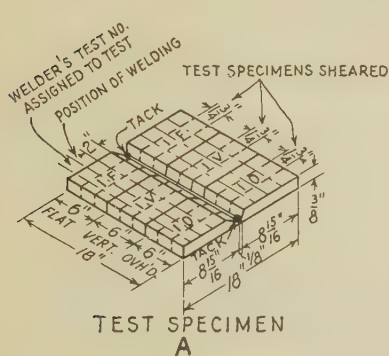
Tensile tests made by various investigators in different parts of the world, and given in Fig. 34, show an average of 38,260 lb. per square inch. This would indicate that a unit stress for arc-deposited metal of 36,000 lb. per square inch, which is used by the author and by other welding engineers, is very conservative for welders who have been trained and tested as outlined.

TABLE 3.—TENSILE TEST RESULTS OF METAL-ARC WELDERS, INCLUDING APPRENTICES

(Base metal used, $\frac{3}{8}$ -in. ship plate. Electrode used, $\frac{5}{32}$ -in. bare low-carbon steel, American Welding Society Specifications, Grade B. Equipment, 1,200-amp. constant potential motor generator set and resistor unit shown in Fig. 25. Requirements, 45,000 lb. per square inch minimum and 52,000 lb. per square inch average. Blank spaces indicate test specimens assigned but not used. No specimens failed entirely in base metal.)

Welder's test number	Arc amperes			Arc volts			Breaking load, pounds			Ultimate strength in pounds per square inch			
	Flat	Ver- tical	Over head	Flat	Ver- tical	Over head	Flat	Ver- tical	Over head	Flat	Ver- tical	Over head	Aver- age
1													
2	175	175	175				29,300	33,280	28,750	52,090	59,168	51,104	54,117
3	165	155	165				33,570	32,250	33,730	59,680	57,328	58,192	58,400
4	160	160	165				30,300	28,820	32,680	53,872	51,232	58,096	54,400
5	155	155	170	13	12	12	33,300	35,650	31,500	59,200	63,376	56,000	59,525
6	160	160	170	12	13	11	34,050	32,300	31,650	60,528	57,424	56,272	58,075
7	165	160	160	12	13	12	31,400	33,850	31,600	55,824	60,176	56,176	57,392
8	160	165	160	13	11	13	36,350	34,600	34,500	64,624	61,504	61,328	62,485
9													
10	165	165	170	11	12	11	28,350	34,150	33,000	50,400	60,704	58,672	56,592
11	160	160	160	11	11	11	31,700	30,300	29,400	56,352	53,872	52,272	54,165
12	155	155	165	14	13	12	30,200	27,900	31,600	55,696	49,600	56,176	53,157
13	155	155	160	13	13	12	33,250	33,800	31,750	59,104	60,096	56,448	58,549
14	160	160	160	14	13	13	30,900	33,100	34,650	54,928	58,848	61,600	58,459
15	160	155	155	15	14	15	34,000	32,300	28,100	61,728	58,624	51,008	57,130
16	165	160	165	11	13	12	33,500	25,750	32,550	59,392	45,776	57,872	54,347
17													
18	160	155	160	13	15	13	32,400	31,750	35,300	57,600	56,448	62,752	58,933
19	155	160	160	14	13	13	37,050	31,900	35,850	65,872	56,704	63,728	62,101
20													
21	155	150	160	14	15	12	32,300	37,000	27,700	57,424	65,776	49,248	57,483
22	165	155	160	11	14	12	35,050	37,500	40,150	62,320	66,672	71,376	66,789
23	155	155	160	14	14	13	34,000	38,550	38,100	60,448	68,528	67,728	65,568
24	160	160	165	12	11	10	36,800	39,200	39,650	65,424	69,680	70,480	68,528
25	155	155	160	14	14	13	36,350	31,400	26,400	64,624	55,824	46,928	55,792
26	160	165	165	12	11	11	40,150	37,200	37,850	71,200	66,128	67,296	66,208
27	155	150	160	14	15	12	38,300	40,300	38,800	68,080	71,648	68,976	69,568
28	160	165	165	12	11	11	35,150	33,300	37,250	62,480	64,528	66,224	64,411
29	160	160	165	12	12	10	36,250	33,150	37,000	64,448	58,928	63,776	62,384
30	155	155	165	13	13	11	26,600	35,900	35,550	47,396	63,824	63,200	58,140
31	155	160	165	14	13	11	31,850	35,850	35,750	56,624	63,728	63,552	61,301
32	155	160	165	14	12	11	34,900	34,750	38,900	62,048	61,776	69,152	64,325
33	155	160	165	14	13	12	34,600	34,050	36,700	61,504	60,528	65,248	62,427
34	160	160	165	12	11	10	37,550	36,900	29,850	61,270	65,600	53,072	59,981
35	155	155	165	13	13	11	32,750	35,050	38,550	58,240	62,320	68,528	63,029
36	155	155	165	13	13	11	38,550	36,350	39,200	68,528	64,624	69,680	67,611
37	155	155	165	13	13	11	33,750	35,700	37,800	60,000	63,472	67,200	63,557
38	155	155	160	13	12	12	35,750	36,300	33,900	63,552	64,528	60,272	62,784
39	150	155	165	14	12	11	36,200	38,150	35,100	64,352	67,824	62,400	64,859
40	155	155	160	13	13	12	35,000	33,250	38,900	62,224	59,104	69,152	63,493
41	155	160	165	14	13	11	37,400	34,950	33,200	66,480	62,128	58,848	62,485
42	155	165	165	13	11	11	37,100	32,650	27,200	65,952	58,048	48,352	57,451
43	155	160	160	13	12	12	37,600	37,750	35,500	66,790	66,752	63,104	65,549
44	155	155	160	14	13	12	35,750	33,350	34,900	63,552	59,280	64,859	62,564
45	150	155	165	14	13	11	38,900	37,450	31,400	69,152	66,576	54,824	63,517
46	155	155	165	13	13	11	33,880	36,300	38,500	60,400	64,528	68,448	64,459
47	155	155	160	14	13	12	36,700	36,900	30,450	65,248	65,600	54,128	61,659
48	150	155	160	14	13	12	29,300	33,650	31,200	52,080	59,824	55,472	55,792
49	155	150	160	14	15	12	29,200	32,750	29,800	51,904	58,224	52,976	54,388
50	160	155	160	12	13	12	36,100	36,300	37,100	64,176	64,528	65,952	64,885
51	155	155	160	13	13	12	33,850	38,000	36,700	60,176	67,552	65,248	64,325
52													
53													
54	155	160	160	13	11	11	38,400	34,950	29,360	68,272	62,128	52,176	60,859
55	150	155	160	13	12	11	33,750	37,050	34,600	60,000	65,872	61,504	62,459
56	155	150	155	12	13	12	35,500	31,750	37,100	62,928	56,448	65,952	61,776
57	150	155	155	13	11	11	36,500	31,300	27,100	64,880	57,448	48,176	56,835
157 2	157 6	162 7	13 15	12 7	11 67	Average.....	60,958	61,389	60,416	60,922			
150	150	155	11	11	10	Minimum.....	47,396	45,776	46,928	53,159			
175	175	175	15	15	15	Maximum.....	71,200	71,648	71,376	69,568			

TABLE 3 (Continued).—DETAILS OF MAKING UP WELDED SPECIMENS FOR TESTING



(j) Using for the welded joint the same factors of safety as now are employed for the riveted joint, basing joint efficiency on the thickness of the thinner plate joined.

With an ultimate strength for arc-deposited metal of 36,000 lb. per square inch and a factor of safety of four, it is possible to obtain a fiber stress for the weld of only 9,000 lb. per square inch. This fiber stress is therefore much lower than the 16,000 lb. per square inch used for riveted construction. The usual factor of safety of four, when applied to riveted structures, is in



FIG. 33.—Welded test specimens.

Minimum, 45,776 lb. per square inch, welder No. 16; maximum, 71,648 lb. per square inch, welder No. 27; average for all tests, 60,922 lb. per square inch. For results see Table 3.

fact only a factor of safety of two, as the strength of the structure is limited by its elastic limit, which is approximately one-half of its ultimate strength. On the other hand, the elastic limit of arc-deposited metal being practically that of its ultimate design strength, the factor of safety of a welded joint is a real factor of four.

The Weakness of Welded Joints under Fatigue and Shock.—

The ability of welded joints to resist fatigue and shock satisfactorily is *primarily conditioned on the use of materials suitable for welding and good design.* Unannealed welded joints in

materials having a high carbon content are inherently defective in these respects owing to the thermal effect in the fusion zone (zone adjacent to the weld metal). On the other hand, steels with a maximum carbon content of 0.25 per cent are satisfactory, as pointed out in Chap. II.

Correct design calls for the lessening of fatigue stresses in the weld by making the joint stiffer than the parts joined. This can be done by the use of straps, reinforcement of the weld, etc. Relative to the endurance limit of welds, Prof. H. L. Whittemore, of the Bureau of Standards, states: "Some tentative results which have been obtained recently indicate that the endurance limits of welds may be about one-half that of the base metal. For low-carbon steel such as is used for buildings, the yield point is about 35,000 lb. per square inch and the endurance limit about the same value. A working stress of 16,000 lb. per square inch is frequently used for buildings. If the endurance limit for welds in such a structure is about 17,000 lb. per square inch, the structure will not fail by fatigue of the welds, particularly as the stresses in a building do not vary greatly nor frequently."

The Westinghouse Electric & Manufacturing Company has demonstrated on full-size specimens that the arc-welded joint resists both fatigue and shock better than the riveted joint. A report on these tests will be found in the September, 1926, issue of the *Journal of the American Welding Society*. Other examples are as follows:

(a) Figure 9 shows a metal-arc-welded 8-in. expansion joint which is used on naval ships. This joint was subjected to an oil pressure of 150 lb., and while under this pressure tested as follows:

Total compression and expansion in both directions	Number of compressions and elongations		Duration of test, hours	Remarks
	Per minute	Total		
$\frac{1}{4}$	$3\frac{1}{16}$	3,309	$17\frac{1}{2}$	No failure, permanent set, or leaks
$\frac{3}{8}$	$3\frac{1}{16}$	1,082	$5\frac{3}{4}$	
Total.....		4,391	$23\frac{1}{4}$	

(b) The 10-ton welded crane shown in Fig. 14 now has been in service for a period of 6 months and is daily subjected to extreme conditions

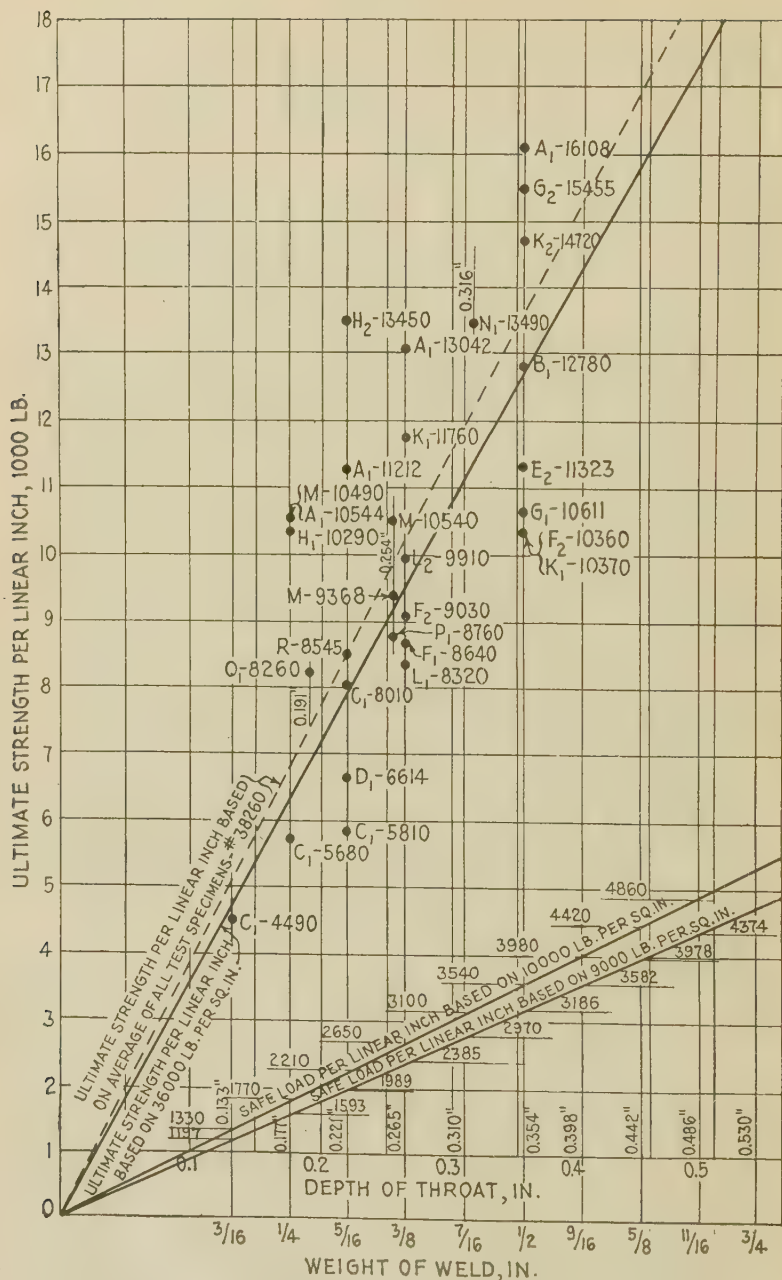
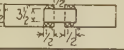
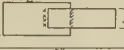
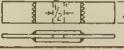
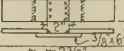
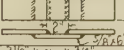
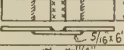
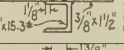
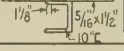


FIG. 34.—Average ultimate strengths of fillet welds

REFERENCE	INVESTIGATOR	SKETCH OF SPECIMEN	NO. OF SPECIMENS TESTED	AVERAGE ULTIMATE STRENGTH WELD METAL LB. IN 2.	ELECTRODE		REMARKS
					ONE LAYER	TWO LAYER	
A ₁	R. A. STORM MORGAN ENG. CO.		30	51175	x		TEST REPORTED BY R. A. STORM IN A.W.S. JOURNAL, JAN., 1926
B ₁	E. S. HUMPHREY, JR. UNION COLLEGE		x	36100	x		TEST REPORTED BY F. P. McKIBBEN AND W. L. WARNER IN A.W.S. JOURNAL, OCT., 1924
C ₁ C ₂	H. E. GROVE METRO GAS CO. MELBOURNE, AUSTRALIA		32 5	32100 36240	Q. A.		TEST REPORTED BY H. E. GROVE IN A.W.S. JOURNAL, JAN., 1927 FIG. 2
D ₁	Do.		30	29930	Q. A.		Do. FIG. 1
E ₂	Do.		12	32000	Q. A.		Do. FIG. 3
F ₁ F ₂	Do.		3 6	32600 31500	Q. A.		Do. FIG. 5
G ₁ G ₂	Do.		3 6	30000 43680	Q. A.		Do. FIG. 6
H ₁ H ₂	Do.		6 6	58200 60000	Q. A.		Do. FIG. 8
K ₁ K ₂	UNION COLLEGE STUDENT		3 3	29320 41603	x		TEST DATA FURNISHED BY W. L. WARNER
L ₁ L ₂	Do.		x x	29320 28000	x		Do. 1 LAYER WELD 3/8" 2 " " 1/2"
M	JAS. W. OWENS N. M. S. & D. D. CO.		3	41500	A.W.S. E-NO. 1B		TOP WELDS 3/8" REINFORCED BOTTOM WELDS 3/16"
N	Do.		2	42600	Do.		TOP WELDS 1/2" REINFORCED BOTTOM WELDS 3/16"
O ₁	Do.		3	43130	Do.		TOP WELDS 1/4" REINFORCED BOTTOM WELDS 3/16"
P ₁	Do.		2	34500	Do.		TOP WELDS 3/8" REINFORCED BOTTOM WELDS 3/16"
R ₁	Do.		2	38000	Do.		TOP WELDS 5/16" REINFORCED BOTTOM WELDS 3/16"

38260

NOTE: BASE METAL OF STRUCTURAL STEEL EXCEPT SPECIMENS "M" TO "R" INCLUSIVE WHICH WERE SHIP STEEL 0.25% CARBON

per linear inch obtained by various investigators.

of vibration and shock, as it is used to lift loads in a foundry cleaning room up to a 25 per cent overload capacity of the crane. To date, there has been no indication of failure.

(c) About 12 years ago the 8-in. pipe connections to the air-receiving tanks at the plant of the Newport News Shipbuilding and Dry Dock Company were arc-welded. Although subjected to vibratory stresses at each pulsation of the pump for this length of time, all joints have remained tight and no failure of welds has occurred.

(d) Service tests of the welded ship "Caria" and the Navy's battle towing targets referred to later.

Low Ductility of Weld Metal and Lack of Homogeneity in the Welded Joint.—An elongation of 5 or 6 per cent for arc-deposited metal is not objectionable, provided suitable joint designs are used and the weld metal in the joint is properly proportioned to take the load; for as pointed out by F. T. Llewellyn, of the United States Steel Corporation: "If we attempted to stretch the steel in any building structure to 20 per cent and reduce its area 30 per cent as required in the specifications for the material used in the structure, the structure would be useless."

Lack of homogeneity of the materials composing the welded joint is also not objectionable for the same reasons. In this connection, Mr. Llewellyn also points out that "a building structure is itself essentially non-homogeneous in that the structural steel is supported on cast-iron base plates, which are in turn supported by concrete, and finally by the earth, piles, etc." Why, then, should we fear for the safety of an arc-welded building when only one of the components of any building (riveted or welded) is ductile and the structure itself is essentially non-homogeneous?

Fixity of Welded Joints Prevents Adjustment under Load.—The assumption that the "come and go" or slippage of a riveted joint is an advantage is a fallacy, for if it were practically and economically possible to erect a structure as a complete unit without one riveted or welded joint, this method of construction would be used as it would be the ideal condition. It follows, then, that a structural joint that makes the parts joined as one piece must be the ideal method of joining metals and not a joint that can slip at 15 to 20 per cent of its ultimate strength, as occurs in the riveted joint.

Lack of Service Tests.—In so far as the application of metal-arc welding to construction is concerned, it is believed that

enough service tests are available to give confidence in this method of construction. A few that I might mention are:

(a) The British ship "Fulagar." This ship was constructed in England by Cammell-Lairds, launched in 1920, and has been in continuous service. On a voyage in 1924 from Manchester, England, to Belfast with a full cargo, she grounded on the sand banks of the upper reaches of the Mersey and at the next rise in tide was floated. Her cargo was discharged and a thorough examination was made of all welded joints by the owners and builders. A report of the accident as given in the *British Journal of Commerce* of July 17, 1924, stated that "she remained fast on the banks during the ebb tide and at low water strained herself considerably, remaining absolutely watertight, although a considerable portion of her length at the bilges was partially set up to an extent of about fourteen or fifteen feet." This report also stated that "if she has been of riveted construction, the joints would doubtless have been opened up and the rivets sheared, and it is doubtful whether she would have been able to get off the bank."

(b) The only joint failures of the battle towing targets (Fig. 10) built by the Navy subsequent to 1922 have been in a lap joint that was provided with a fillet weld on only one edge and in a double-lap joint in which the opening between the two edges was two and a half instead of five times the thickness of the thinner material joined, and which recent tests show to be the minimum necessary. These failures were owing to grounding on a coral reef.

(c) Service tests on the welded British cross-channel barge built in 1920, several sea-going tugs, barges, etc., all indicate that these structures have given entire satisfaction.

Porosity of the Weld.—Metal-arc welds made in mild steel, by properly trained operators, on surfaces free from oil and grease, will not show any appreciable porosity. Where slight porosity does occur, it can readily be stopped by local peening under test. Some makes of coated electrodes have been found to reduce porosity.

Corrosion of the Weld.—At each docking of the Navy's battle towing targets an examination has been made to ascertain the condition of the welds. In no case have they been found to be corroded to any greater extent than the base metal. The only case of serious corrosion known to the author is a local corrosion of a weld in the bilge keel of a naval ship. This weld was located immediately below an ash ejector, and corrosion in this case was caused by the sulphurous content of the ashes. The shell plating in the vicinity of the weld also was corroded badly

from this cause. Numerous weld specimens have been submerged at tide level and attached to the hulls of ships for periods of over a year and no appreciable corrosion has been noted.

Arc Welding Not Suitable Where Power Supply Is Not Available.—As gasoline-driven motor-generator arc-welding sets are now standard commercial equipment, such sets with the necessary gasoline supply usually can be transported readily. This objection is therefore not pertinent.

Difficulty of Repair.—When steel is used instead of cast iron, repairs are made easily and cheaply by the arc, as no preheating is required and the repairs can usually be made in place.

In case of damage to large steel structures, such as a ship's hull, repairs also can be made quickly either by chipping out the welds with a pneumatic chipping hammer or by cutting out the damaged part with the carbon arc or the gas torch. Instead, therefore, of repairs being more difficult or expensive to make on welded structures, it will be found that the reverse is true.

That Welding Costs More than Riveting.—Riveting has been used for years in construction work and has been found in general to be satisfactory. Justification for replacing this method of construction with another must therefore be based on lower cost, the production of a better product, the elimination of noise, or some other good reason. As it has been shown in Chaps. I and II that arc welding not only produces a better product, but eliminates noise, and that lower costs can be secured consistently by its use, no further consideration will be given to this phase of the subject. When, therefore industrial organizations have, under the guidance of competent welding engineers, acquired a few more years of experience with the process and have proved, as has the author, that material, labor, and overhead costs of arc welding are in general less than those of riveting, its applications will materially encroach on riveting.

CHAPTER IV

DESCRIPTION OF SOME RECENT OUTSTANDING APPLICATIONS OF ARC WELDING

So far there have been considered the advantages of arc welding, its fundamentals as applied to design and shop practice, and a discussion of the stock objections to its use. The next step is to show how these advantages have been utilized, the fundamentals applied, and the objections overcome. This chapter will therefore be devoted principally to a description of four recent outstanding arc-welding applications at the plant of the Newport News Shipbuilding and Dry Dock Company and one of them a project on which that company as a subcontractor furnished 72,600 tons of steel, *viz.*:

(A) The 90-mile pipe line for the East Bay Municipal District in California.

(B) The Coast Guard cutter "Northland."

(C) The S.S. "California."

(D) The S.S. "Virginia."

(A) **The 90-mile Pipe Line for the East Bay Municipal District in California** (Figs. 5, 35, and 36).—The principal reasons for using welding on this project were to *reduce the cost of construction and to reduce the leakage*.

This pipe line, which is practically complete, is undoubtedly the largest undertaking in the history of arc welding. It will supply water from the Mokelumne River to Oakland and the other cities east of San Francisco Bay. It is approximately 90 miles long and is constructed of 30-ft. pipe sections 58 to 65 in. in diameter and of $\frac{3}{8}$ -, $\frac{7}{16}$ -, and $\frac{1}{2}$ -in. material. Each section has two longitudinal butt joints welded by the Lincoln automatic carbon-arc machine shown in Fig. 26,¹ and all were hydraulically tested in the shop, on completion of welding, to a pressure slightly below the elastic limit of the material. While under this pressure, each joint was subjected to a series of blows

¹Just prior to the completion of this project, the longitudinal joints of approximately one mile of line were welded by an automatic metal-arc machine, similar to that shown in Fig. 27.

from 19-lb. hammers placed 12 in. apart along the seam and dropped through an arc of 4 ft. A test of such 'severity' is unheard of in riveted construction, yet all sections installed in the line successfully passed the test. Three methods were used for connecting the 30-ft. sections in the field—riveting, gas welding, and metal-arc welding. The longitudinal carbon-arc-welded joints are shown in Figs. 5 and 35 and the preferred metal-arc-welded circumferential joint in Fig. 35.

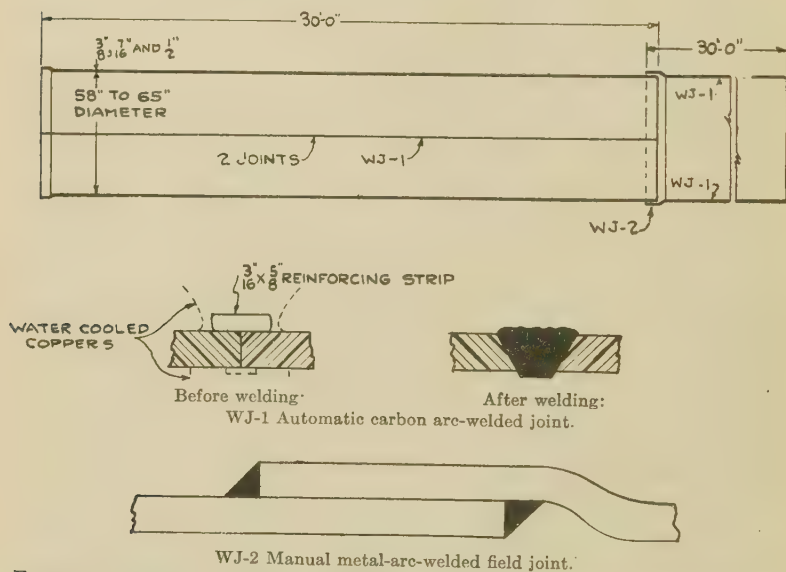


Fig. 35.—Details of carbon- and metal-arc-welded joints used in the 90-mile pipe line for the East Bay Municipal Utility District of California.
No welds failed under hydrostatic field test.

The published bids submitted for riveted and for welded construction by the successful bidder for 83.7 miles of the line were, respectively, \$11,354,278 and \$9,236,996—a difference of \$2,117,282, or 18.6 per cent in favor of welded construction.

The estimated savings in weight of material required for riveted construction *versus* the material required for welded construction, as obtained from these bids, was also 18.6 per cent. There actually was used on the project approximately 72,650 tons of steel. These savings were made possible by:

(a) The higher efficiency of the welded joint, which permitted a decrease in the thickness of the pipe.

(b) A reduction of 2 to 3 in. in the diameter of the pipe by the elimination of rivet heads, which increase friction.

(c) The lower cost of labor involved in welded construction.

Messrs. Jones and Weeks have stated in "A Physical Study of the Mokelumne Pipe Line," recently published by the University of California, that "if this pipe line was riveted, it might have a maximum permissible leakage sufficient to supply a city of more than 10,000 population," whereas all welded joints tested, in the shop and after installation, were free from leaks. Figure 36 shows a 30-ft. section of this pipe being lowered into a trench.

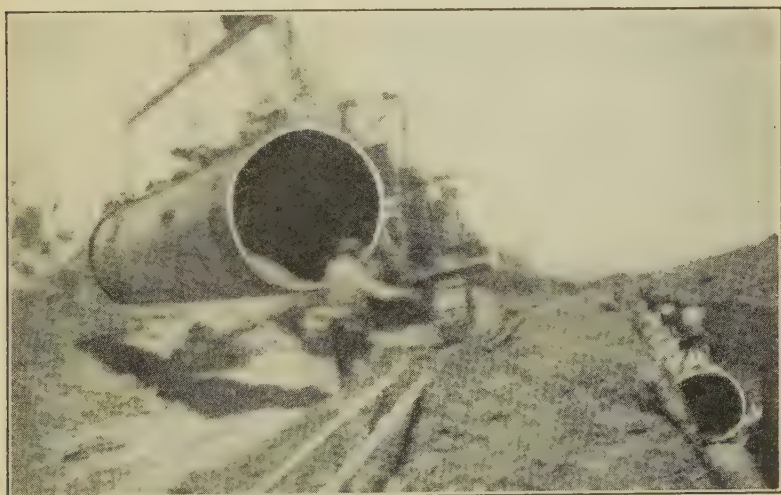


FIG. 36.—Lowering section of arc-welded pipe in trench.

(B) The Coast Guard Cutter "Northland" (Figs. 37 to 44).—The Coast Guard's primary reason for permitting the extensive use of welding on the "Northland" was *to obtain a stronger ship than one entirely riveted*, the ship being designed to be the sturdiest ship ever constructed.

As the application of welding to this ship was not taken up with the Coast Guard Service until the drawings of the ship were well advanced, it was impossible to use welding to the extent that it might have been or to apply it in every case to the best advantage. In so far as the contractor is concerned, its utilization made possible the most extensive application of welding

to ship construction that ever had been attempted in the United States.

The "Northland" was launched Feb. 5, 1927, and replaces the historic ship "Bear" for service in the Arctic ice floes north of Alaska and Siberia. She already has been through several bad gales, and no welded joints have failed. It has an overall length of 216 ft., a breadth of 29 ft., and a depth of 52 ft. The seams and butts of its decks and superstructure are very



FIG. 37.—The Coast Guard Cutter *Northland*.

First ship built in United States in which arc welding was extensively employed to increase its strength, 13,000 lb. of electrodes being used.

largely welded, and a $\frac{5}{8}$ -in. continuous fillet weld is used to reinforce and make tight the double-riveted seams of four $\frac{7}{8}$ - to $1\frac{1}{4}$ -in. strakes of its side plating at the waterline, *where extraordinary strength and stiffness are needed to resist ice pressure*. Her rudder (Figs. 41 and 44) was constructed by having $\frac{5}{8}$ -in. plating welded to a cast-steel frame. This rudder weighs approximately $4\frac{1}{2}$ tons. Her stern frame, also shown in Figs. 41 and 44, was made in two parts, annealed, and joined by thermit welding. A total of 13,000 lb. of electrode material was used on this ship.

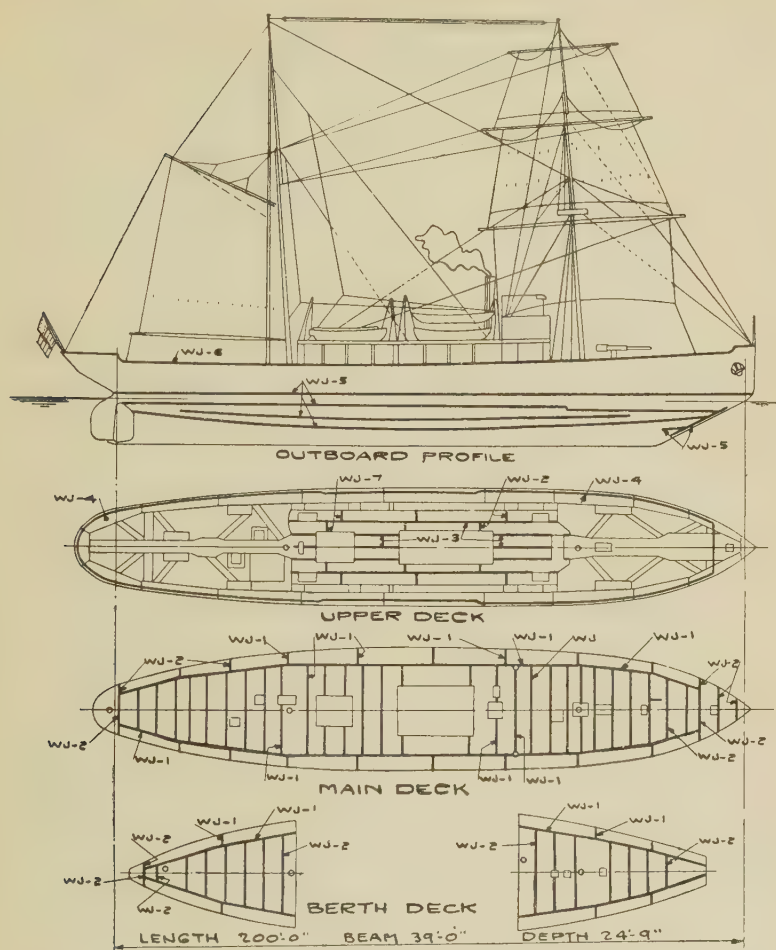


FIG. 38.—Welding of shell and decks of Coast Guard cutter *Northland*.

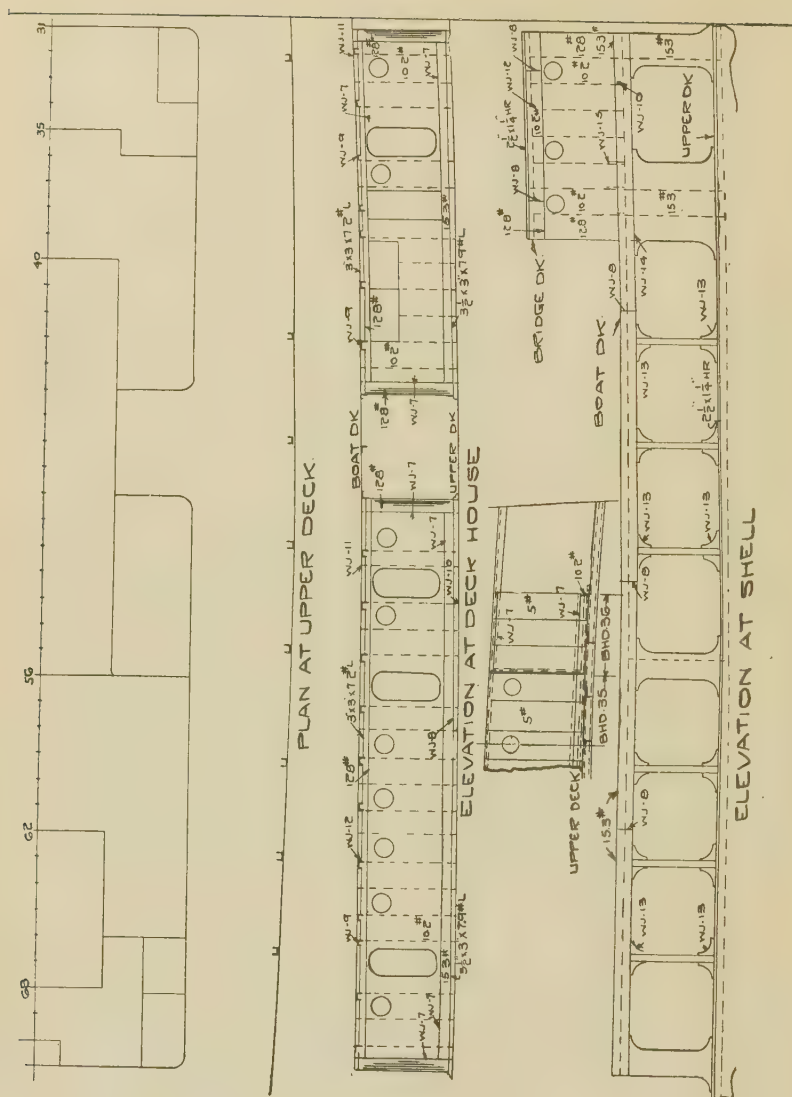


FIG. 39.—Welding of bulkheads of Coast Guard cutter *Northland*.



FIG. 40.—Details of metal-arc-welded joints of Coast Guard cutter *Northland*.

Later tests have proved the advisability of locating edges to be welded at least five times thickness of thinner plate apart.

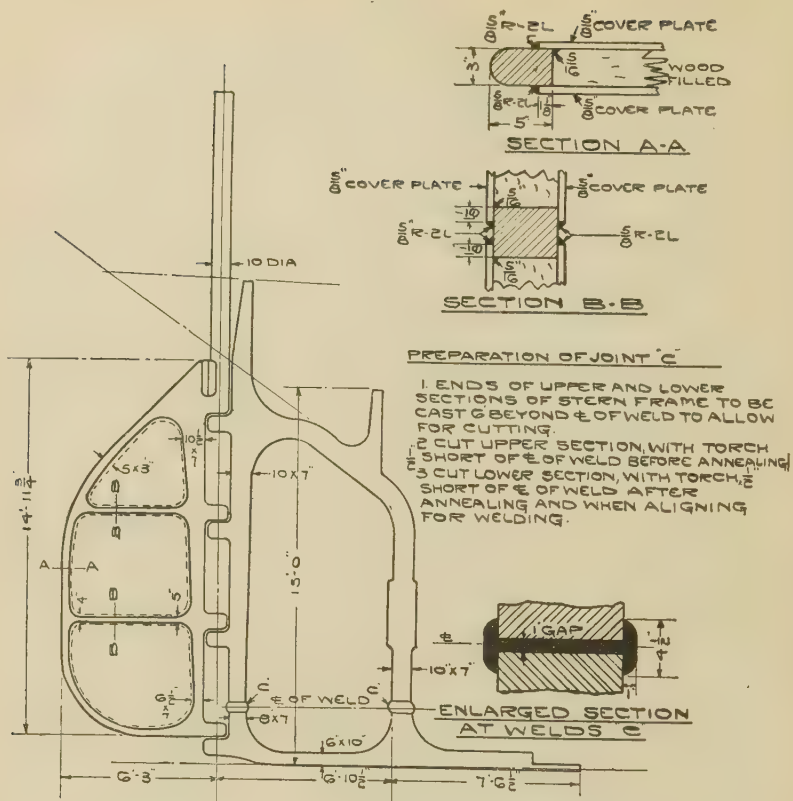


FIG. 41.—Arc-welded rudder and thermite-welded stern frame of Coast Guard cutter *Northland*.



FIG. 42.—Arc-welded seam and butt of deck of Coast Guard cutter *Northland*.



FIG. 43.—Arc-welded superstructure of Coast Guard cutter *Northland*.

(C) **The S.S. "California"** (Fig. 45).—The advantages of welding in this case were *flexibility and tightness*, for its use obviated the holding up of plans for the location of bulkheads, permitted a flexible arrangement of them, and avoided the punching of the deck plating in oiltight and watertight compartments (Appendix Figs. 44-C and D).

This ship is the largest merchant ship constructed in the United States to date and was launched on Oct. 1, 1927. It is

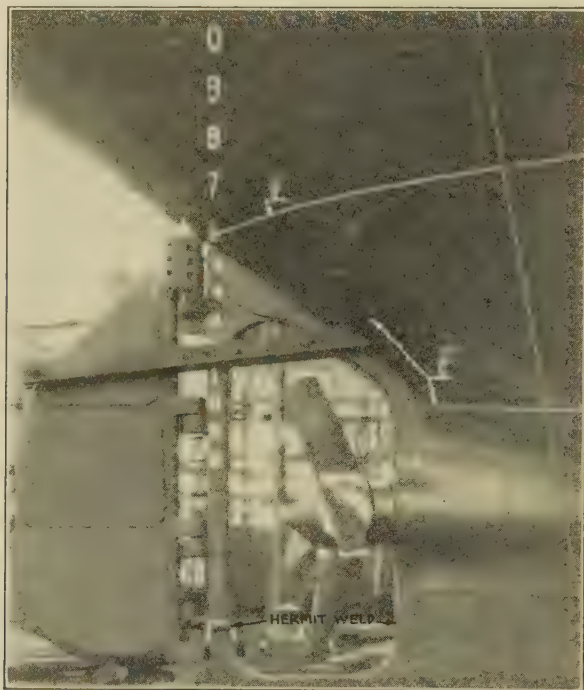


FIG. 44.—Stern of Coast Guard cutter *Northland*, showing welded rudder, stern frame, and shell seams.

a twin-screw passenger and freight ship owned by the Panama Pacific Line. It has an overall length of 601 ft. 3 in., a breadth of 80 ft., a depth of 52 ft.; and the principal application of welding was the attachment of all partition bulkheads to decks, using in most cases a plate coaming. The total quantity of electrodes used was 30,000 lb.

(D) **The S.S. "Virginia"** (Figs. 46, 47, and 48).—The "Virginia," a sister ship of the S.S. "California," which is now

under construction, will be 12 ft. longer, and welding will be used much more extensively than on the "California," so as to *construct a better ship, to increase its deadweight carrying capacity, and to reduce the cost of construction and maintenance.* It is probable that at least 45,000 lb. of electrodes will be employed.

The additional welding which has been approved to date by its owners and the American Bureau of Shipping is as follows:

(a) Composite frame splices shown in Fig. 46. This connection avoids the lapping of members as in present practice, the necessity

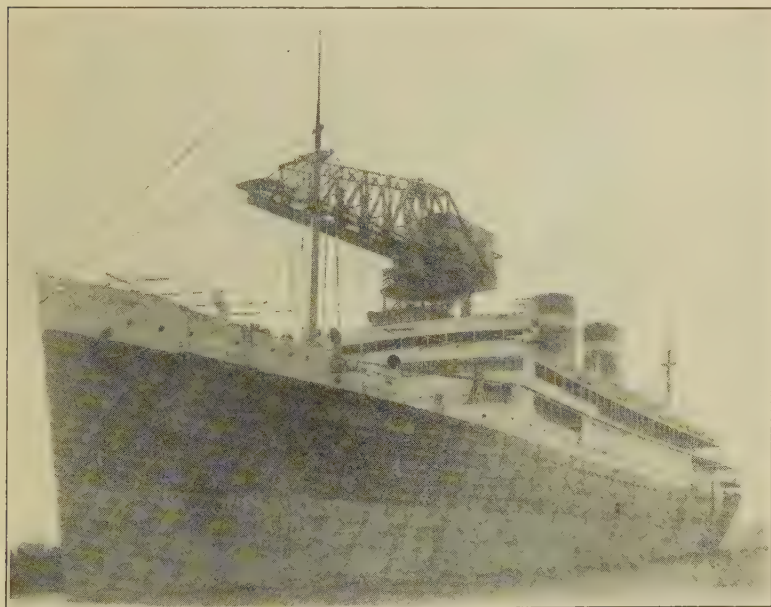


FIG. 45.—S.S. *California*.

for offsetting frame rivets, and secures a reduction in the weight of the connections and their cost. A test specimen of a similar connection is shown in Fig. 47.

(b) Tops and bottoms of all pillars as shown in Appendix (Fig. 28). This connection was developed on the basis of securing an ultimate weld strength twice that of the shearing value of the riveted connection. Its cost is approximately one-quarter that of the usual fire-worked and riveted angle connection.

(c) Natural ventilation ducts throughout the ship and approximately 95 per cent of their connections to bulkheads and decks (Appendix, Figs. 37 to 43).

(d) Corners of all boundary angles and connections in the way of joggled seams and butts as shown in Appendix, Figs. 26 and 27, all welding being done on the ship.

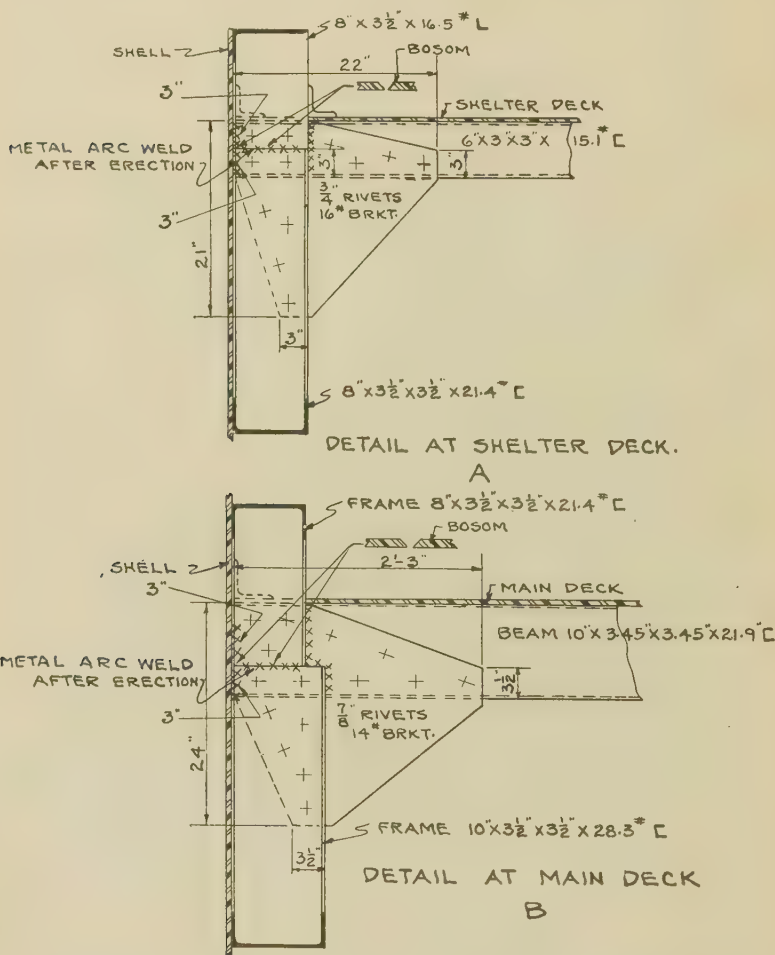


FIG. 46.—Composite frame splice.

NOTE: (1) Approved by American Bureau of Shipping for S.S. *Virginia* for all frames;
(2) All fillet welds $\frac{3}{8}$ in.

The American Bureau of Shipping approved the complete welding of its masts, as shown in Fig. 49; approval, however, was not obtained from the owners. It will be noted on this drawing that bar connections have been substituted for mast and boom bands, and that wherever practical these bars pass entirely

through the boom or mast. This method of attaching fittings, etc., to masts and booms is an adaptation of airplane joint design and is not only superior to the present band type of connection but is very much cheaper. The weight of this mast is 18.2 per cent less than that of the corresponding riveted mast used, and its cost would have been approximately 15.7 per cent less.



FIG. 47.—Bend test of a composite frame splice similar to those shown in Fig. 46.

Arc-welding Applications Adopted as Standard Practice.—

Many of the arc-welding applications to ship construction referred to have been adopted as standard practice by the Newport News Shipbuilding and Dry Dock Company. Three other important applications to ship construction are worthy of note in this connection:

(a) The welding of deck beams directly to bulkheads and the use of welded plate collars instead of riveted stapling in the way of deck beams and bulkheads. These methods of attaching deck beams to bulkheads were used on the Coast Guard cutter "Northland," the

S.S. "California," the "Algonquin," and on other ships (Fig. 48, and Appendix, Fig. 25).

(b) All main condenser shells (Fig. 15). Completely welded condenser shells have been installed on the S.S. "Caracas," "Algonquin," and "Yorktown."

(c) Rudders (Figs. 41, 44, and 65). Completely welded double-plate rudders have been used on the Coast Guard cutter "Northland" and the S.S. "Yorktown."

(d) All black-sheet-metal tanks not a part of the hull structure and all galvanized tanks that can be made of black metal and welded prior to galvanizing (Appendix, Fig. 55).

Summary of Arc-welding Applications to Date.—In the Appendix are listed all welded parts of ships recently constructed

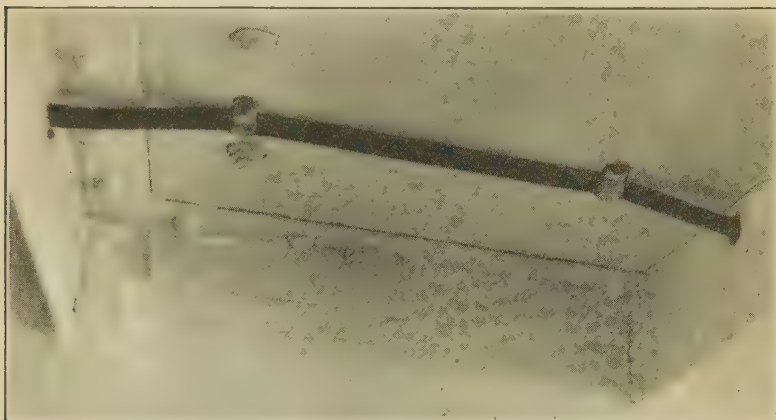


FIG. 48.—Deck beams welded to bulkhead on S.S. *Algonquin*.

at the plant of the Newport News Shipbuilding and Dry Dock Company or now under construction. No attempt has been made to list recent applications of other shipyards or of the Navy, although the preliminary sketches and specifications for welding on the new scout cruisers were prepared under the author's direct supervision.

Policy Pursued in Overcoming Stock Objections.—All of the ships built at the Newport News shipyard during the past two years and on which welding has been extensively used either have come under the rules of the American Bureau of Shipping or are not classified. In the case of the Coast Guard cutter "Northland" it was necessary only to convince the Coast Guard Service of the advantages of arc welding and to assure

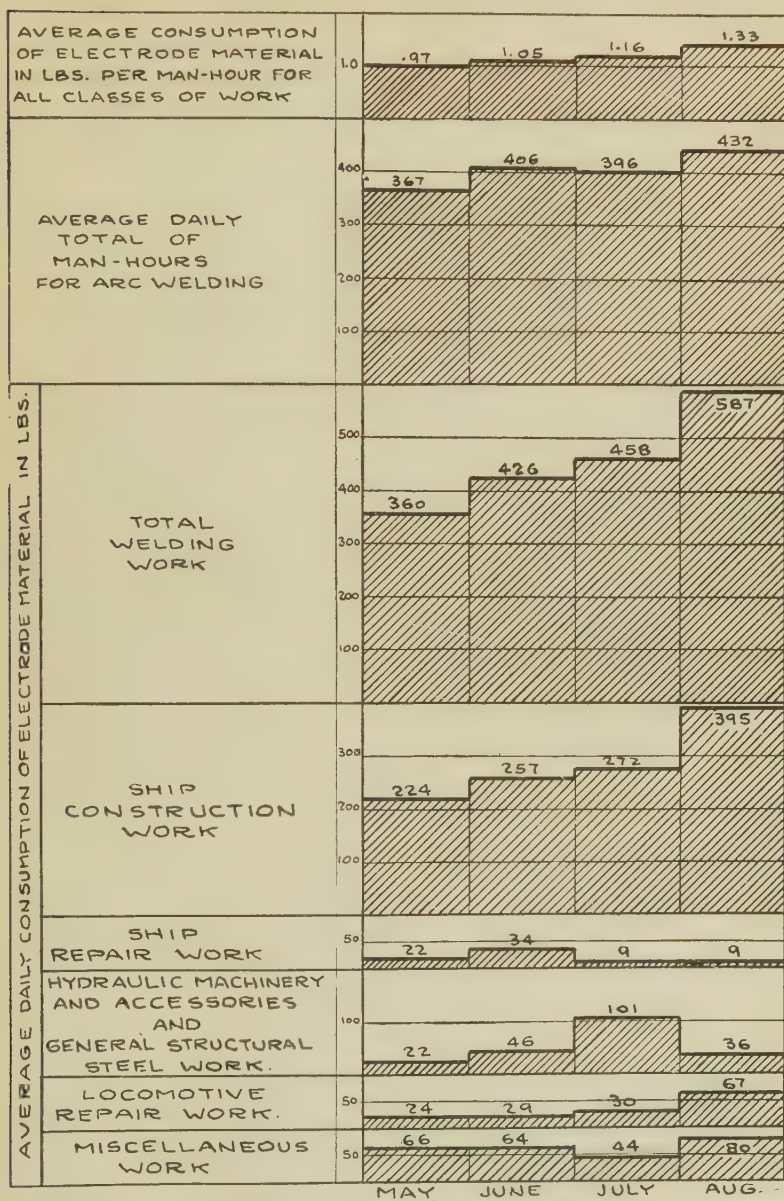


FIG. 50.—Curve showing the relative amount of arc welding on the various classes of work at the plant of the Newport News Shipbuilding and Dry Dock Company and the average deposition rate per man hour.

it that good workmanship would be obtained. In the case of the S.S. "California" and S.S. "Virginia" and other ships constructed under the rules of the American Bureau of Shipping, it was necessary to conduct an educational campaign among their owners, architects, and inspectors to obtain approval of the welding desired, and finally approval of the plans by the American Bureau of Shipping.

It should be stated that the American Bureau of Shipping has shown an excellent spirit of cooperation in the development of welding and during the past year has entirely revised its rules, incorporating therein a new section covering the application of welding to ship construction, which will make possible the construction of completely welded ships.

Figure 50 shows the relative amount of arc welding being done on the various classes of work at the plant of the Newport News Shipbuilding and Dry Dock Company and the average deposition rate per man-hour on all classes of work. It is interesting to note the upward trend of the "work" curves owing to the yard's campaign to increase the use of welding. The gradual improvement in the metal deposition rate is owing to the installation of better equipment and to improved supervision and technic in the welding shop.

CHAPTER V

COMPARATIVE RIVETED AND WELDED DESIGNS OF A LARGE FREIGHT SHIP

A ship has been selected to illustrate the economic advantages of arc welding for several reasons:

First.—With the exception of aircraft and rolling stock, the savings obtained by arc welding are in general restricted to first cost. The weight saved in a ship's hull and machinery, on the other hand, makes possible a greater carrying capacity, or a reduction in the cost of operation, or an increase in speed, any one of which advantages has a bearing on net income throughout the life of the vessel. In naval ships the weight saved permits an increase in armor, armament, speed, and cruising radius; these military characteristics always have been important, but now weight is important more than ever before owing to the recent treaties limiting the displacement of naval vessels.¹

Second.—A ship's structure presents more difficult problems of fabrication and erection than any other type of heavy construction owing to the curved shape of the hull and the necessity for watertight and oiltight work.

Third.—A ship's hull in service is subjected to stresses that are more complex than those of any other structure (excepting possibly aircraft) on which both arc and gas welding are now extensively used for strength joints.

Fourth.—If it can be shown, as the author has endeavored to do in the foregoing chapters and will enlarge upon further in this chapter, that arc welding is now being, and can be, practically and economically applied to a ship's hull (a structure that is fabricated from steel, the most extensively used material in industry; a structure that is difficult to assemble; and a structure that is subjected in service to complex stresses), then the universal utilization of the arc-welding process is assured.

¹ The weight saved by the extensive use of welding in the construction of the new German cruisers is largely responsible for the mounting of 11-in. guns on these ships.

The choice of an ocean freighter over a passenger ship, a lake freighter, etc., is owing to the fact that over 80 per cent of the world's ship tonnage falls within this class, a freighter of the size selected being typical of modern design.

General Design.

Both Riveted and Welded Ships.—The following figures and particulars show the outboard profile and the important features of a typical, modern, moderately fast, single-screw, three-deck, steel ocean freight ship:

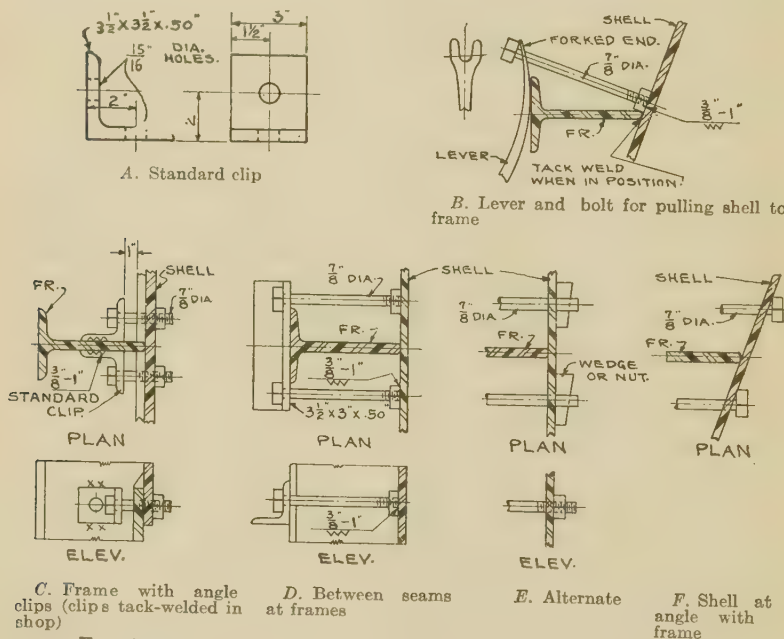


Fig. 51.—Assembly devices for welded ships and other structures.

Length overall, 442 ft. 0 in.; length between perpendiculars, 425 ft. 0 in.; breadth molded, 57 ft. 0 in.; depth molded to upper deck, 42 ft. 0 in.; maximum draft, 31 ft. 8½ in., with 17,170 long tons displacement; working draft, 28 ft. 0 in., with 14,990 long tons displacement; main machinery, one 3,500-i.hp. triple-expansion engine; three single-ended Scotch boilers 16 ft. 6 in. × 12 ft. 0 in.; speed, 12 knots on trial, or 11½ knots sea average. The riveted design scantlings conform to the highest rating of

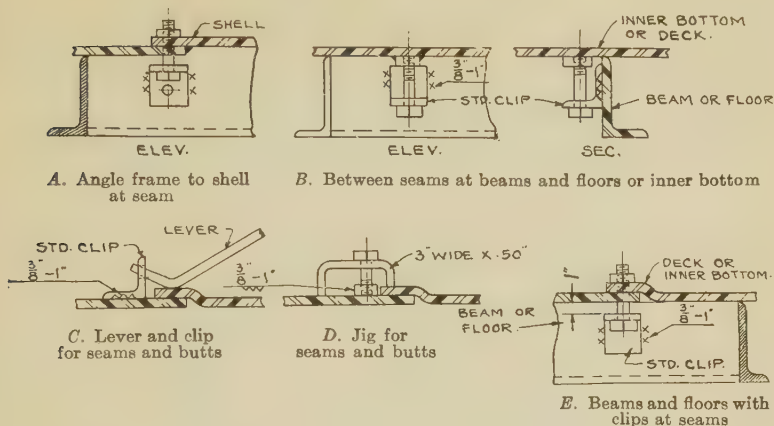


FIG. 52.—Assembly devices for welded ships and other structures.

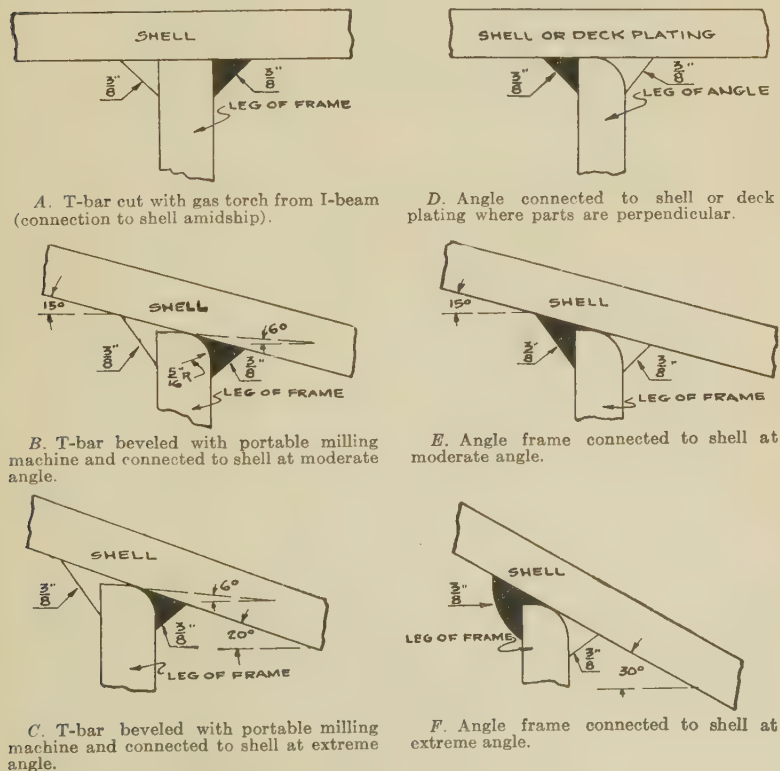


FIG. 53.—Connection of frame and deck beams to plating.

NOTE: All fillet welds $\frac{3}{8}$ " 3" 12" S.

the Classification Societies, and the welded design is of equivalent or greater strength.

The Welded Ship.—The same outboard profile (Fig. 58) has been used for the arc-welded ship, but the details of its structural design are so radically different from those of the riveted ship

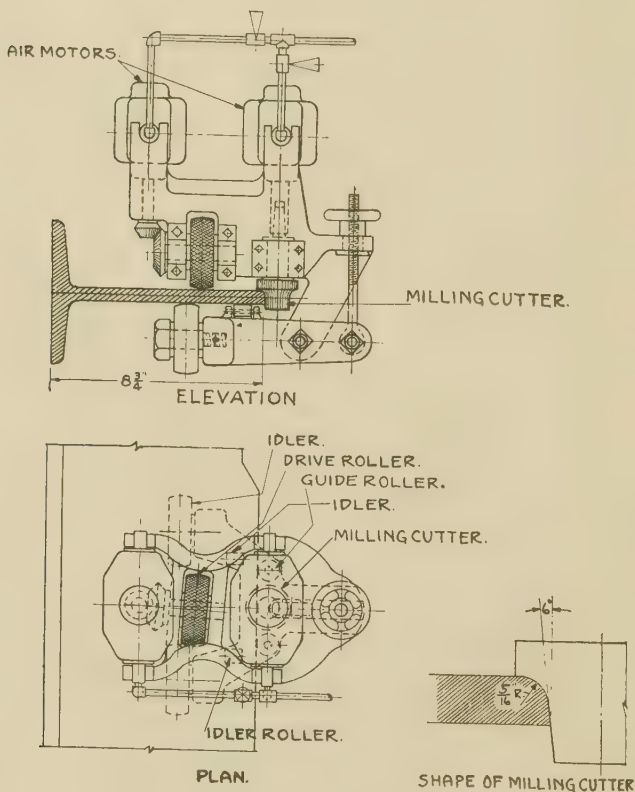


FIG. 54.—Proposed portable edge-milling machine.

that it has been necessary to prepare independent drawings for comparative purposes and to illustrate clearly its principal design and assembly features (see Figs. 53, 55, 56, 57, 60, 62, 63, 65, 67, and 69). The principal dimensions and arrangement of the riveted ship have been adhered to strictly for the welded ship.

The Tentative Standards for Ship's Hulls and Machinery, in the Appendix, cover the majority of the design features of the

welded ship not shown in the figures referred to, and specific reference will be made to them.

In general there have been employed throughout the welded ship only those features of design that would make its construction possible under the existing welding requirements of the American Bureau of Shipping and the Lloyds Register of

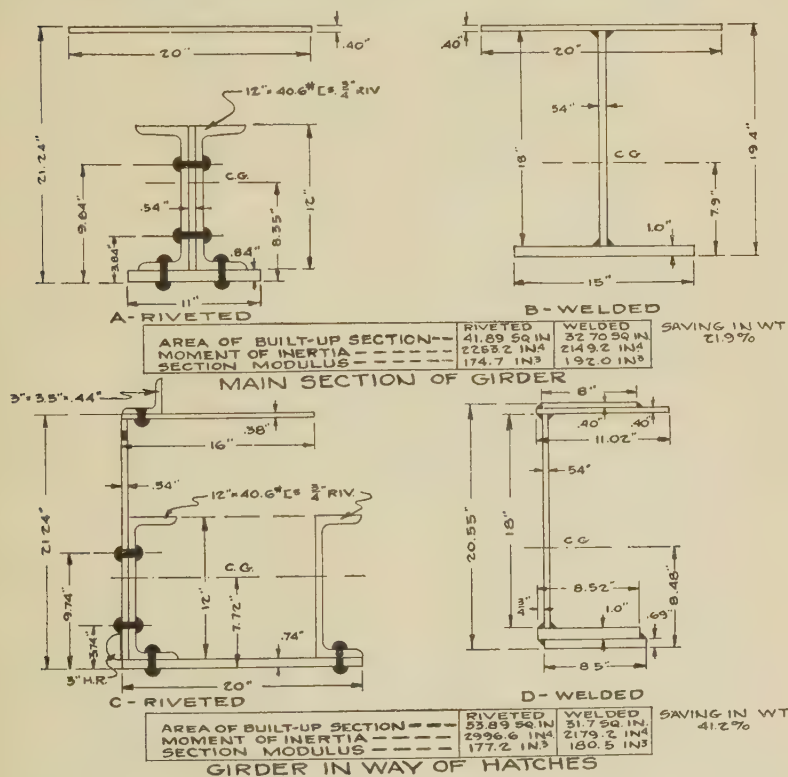
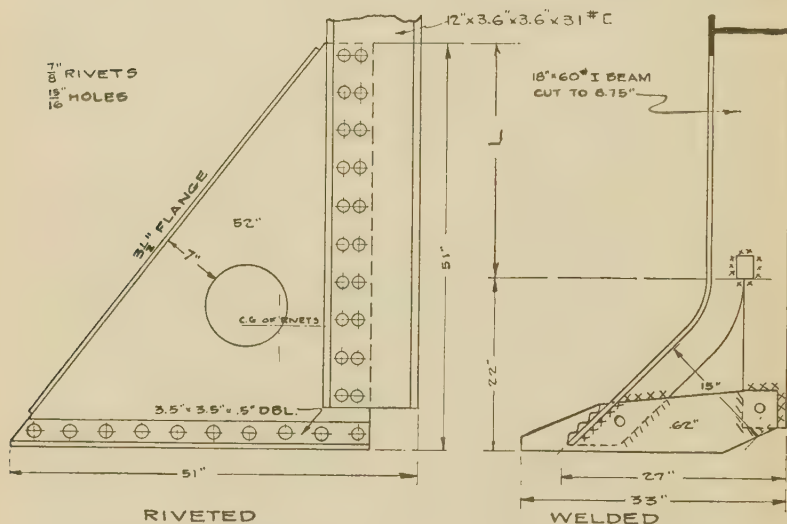


FIG. 55.—Comparative designs of longitudinal deck girders for riveted and welded freight ships.

Shipping. The Symbols, Nomenclature, Fundamental Specifications, and Design Data, given in the Appendix, have been adhered to consistently. The efficiencies of the corresponding riveted and welded joints are in many cases given on the drawings. All welded joints have been made stronger than the corresponding riveted joints.

The principal changes from the riveted-hull design are:

(a) A reduction in the weight of the frames; deck beams; bulkhead stiffeners and plating; deck girders and deck plating inboard of stringers; inner-bottom plating, except center-line and margin plates; solid and partial floors; rudder; hawse pipes; etc. Where a reduction has been made in the weight of bulkhead, deck, and inner-bottom plating, such reduction has



WEIGHT OF BRACKET-----	RIVETED	WELDED
ULTIMATE RESISTING MOMENT OF FRAME	339 #	97 #
VALUE OF BRACKET IN COMPRESSION--	2,420,000" #	2,420,000" #
VALUE OF BRACKET IN TENSION--	2,640,000" #	3,110,000" #
	4,650,000" #	5,110,000" #

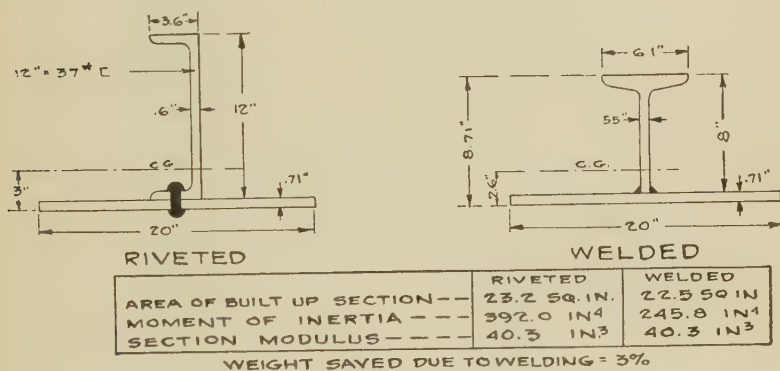
SAVING IN WEIGHT OF WELDED BRACKET OVER
RIVETED BRACKET 71.4%

Fig. 56.—Comparative designs of frame brackets for riveted and welded freight ships.

been limited to $\frac{1}{16}$ in. (0.0625 in.) as it is believed that permission can be secured from the Classification Societies to do this, the weight of plating specified for the riveted design being largely determined by the mechanical requirements for caulking. In so far as strength is concerned a further reduction would be possible, but in order to secure the approval of the Classification Societies it is believed that a $\frac{1}{16}$ -in. reduction of the parts previously mentioned is as far as we should go at the present

time. Reduction in the weight of sections has been made possible by the substitution of more efficient compound sections in place of the less efficient riveted ones and/or the higher efficiency of the welded joints used.

(b) The elimination of angles and other flanged connections such as is consistent with good welded design and practical construction methods.



COMPARISON OF MAIN FRAMES

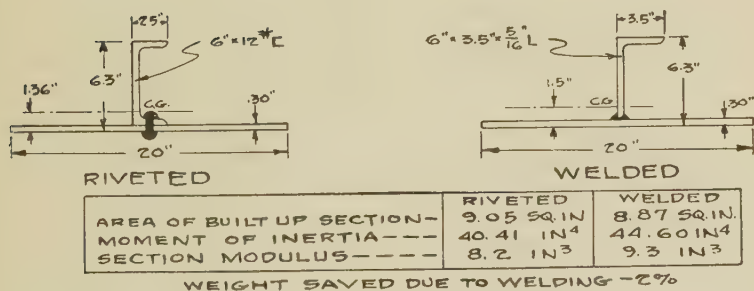


FIG. 57.—Comparative designs of main frames and bulkhead stiffeners for riveted and welded freight ships.

(c) The lap of all double, treble, and quadruple riveted joints has been reduced approximately to that of a single-riveted joint, without sacrificing the original efficiency of joints required by the rules of the Classification Societies for riveted ships. A further reduction in the width of lap is possible in many cases, especially where no assembly holes are required.

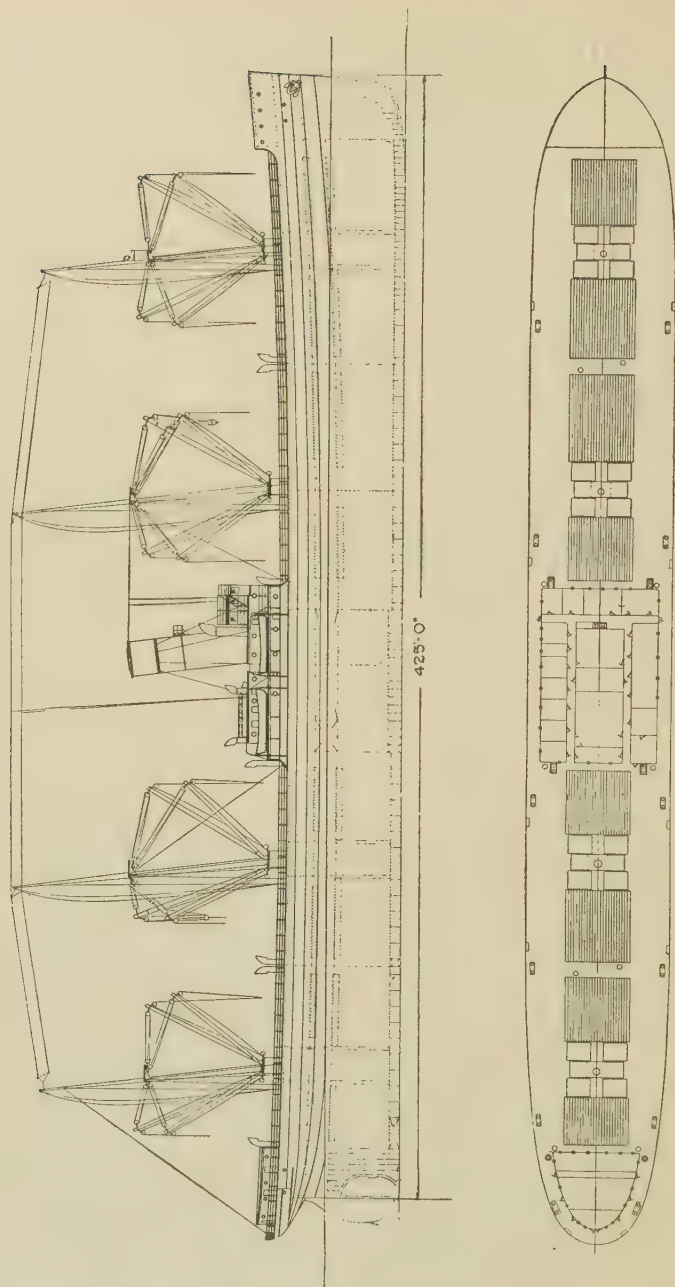


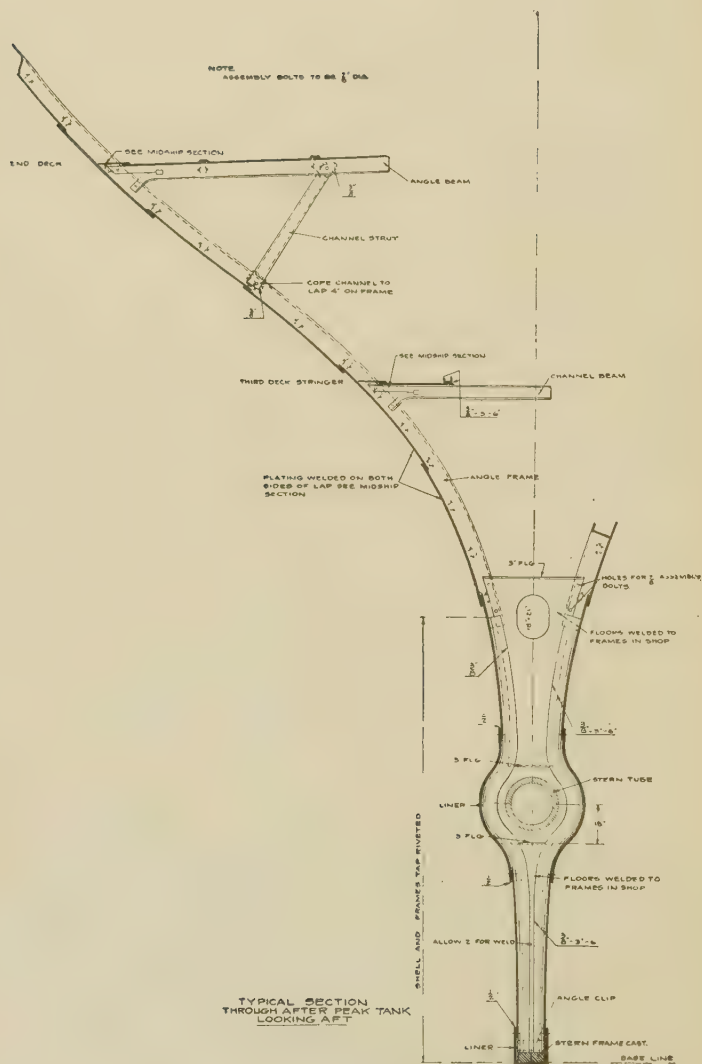
FIG. 58.—Outboard profile of 12,720-ton dead-weight riveted freighter and of 13,220-ton dead-weight welded freighter.

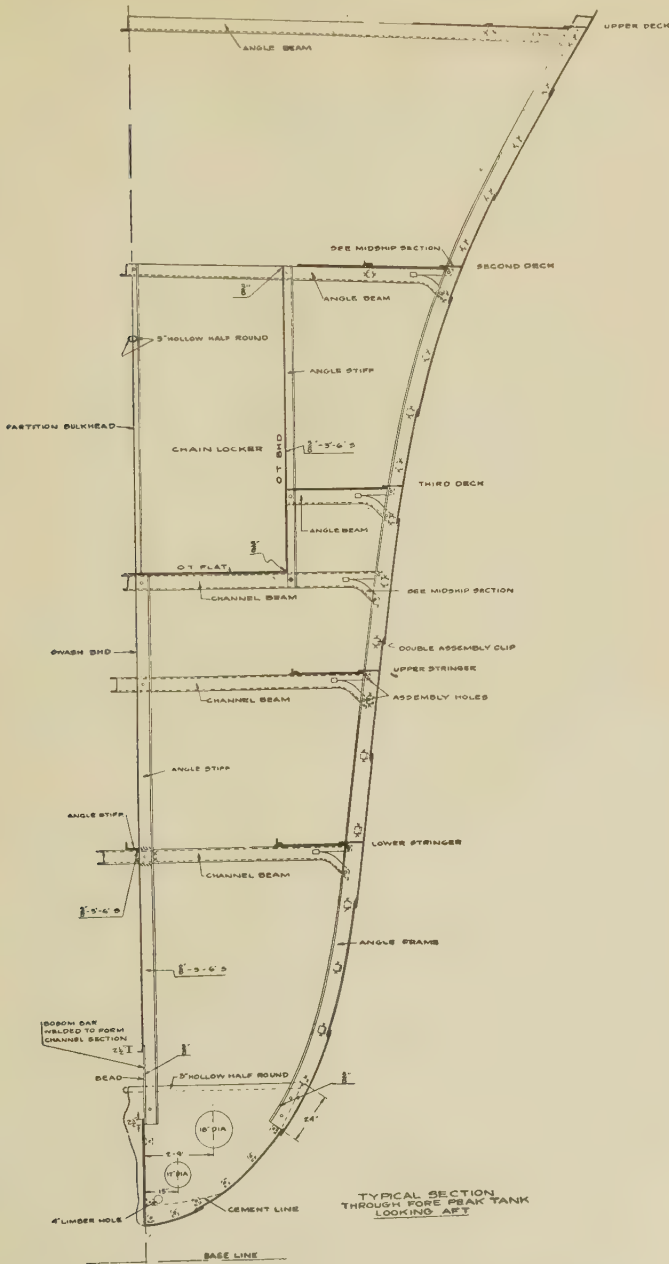
Subassembly and Welding.—It is proposed to subassemble and automatically weld, without the use of assembly holes, all possible parts of the ship; for example, frames, girders, bulkheads, foundations, partial and solid floors, rudder, pillars, and skylights. It also is proposed to employ automatic metal-arc welding during subassembly and on the ways wherever possible. A proposed automatic welding machine for bulkheads is shown in Fig. 28 and described in Chap. II.

Assembly Devices and Welding Methods (Figs. 51 and 52).—As experience has shown that it is not practical to construct and “fair” a ship without holes for the location of parts and to pull the faying surfaces together, this method of assembly has in general been employed for the erection of the welded ship on the ways. The size of holes has been made $1\frac{5}{16}$ in. throughout the ship, except those used for the bolting up of the shell to stem and stern post, which are $1\frac{3}{16}$ in. The number of holes has been reduced to the very minimum, by spacing them widely apart on straight surfaces and by the extensive use of assembly nuts and clips. Clips are punched on a production basis and tack-welded or bolted to one of the members to be joined prior to erection. Nuts, when used, are tack-welded to the member, either prior to or during erection. The maximum spacing of holes is approximately that of the frame spacing, which in the case of this ship is 2 ft. 10 in., or approximately 3 ft. This spacing is halved, quartered, or further subdivided as necessary on curved or other surfaces difficult to assemble and is an important feature in cost reduction. If the holes provided are not sufficient to pull the faying surfaces together, either additional holes can be cut on the ship with a torch or nuts can be tack-welded to a member for the use of suitable clamps, clips, etc.

Where oil-tightness, water-tightness, or appearance are not required, assembly holes need not be filled; however, when tightness or appearance is necessary, rivets are driven or the hole is filled with weld metal, depending upon conditions. When rivets are used to fill the assembly holes, their presence has not been taken into consideration in calculating the efficiency of the joint, except in the attachment of shell plating to stem and stern frame (Fig. 60, Detail WJ-11).

It is not the intention to describe or to show in detail on the drawings all of the necessary assembly holes, clips, nuts, etc.,





welded 13,220-ton dead-weight freighter.

that are necessary for assembly purposes, but merely to describe and to illustrate the methods so as to show their practicability. Again, it is not the intention to punch and tack-weld assembly clips and/or nuts on the same member, except when holes are absolutely necessary or when the use of assembly clips and nuts will reduce cost. If desired, the tack-welded clips and nuts can be knocked off readily with a sledge after they have served their purpose.

Details of Design, Fabrication, Etc.

Keel (Fig. 60, Detail WJ-6).—The angles connecting the vertical keel to the shell and inner bottom have been eliminated, and this material has been put into the plates. Individual sections, approximately 40 ft. long, may be dogged down on a welding slab, without the use of assembly holes, and the maximum amount automatically welded, as shown at *A* in Fig. 28, or the keel may be completely assembled on the ship and automatically welded.

These sections will be joined on the ways by joggled lap joints on the web and a strapped butt joint on the flat plate keel. A joggled lap will also be used for the center-line strake of the inner bottom. Assembly holes are provided in straps and joggled laps. Assembly clips are tack-welded on the web to take the floors and partial floors which are provided with assembly holes.

Shell (Fig. 60, Detail WJ-4).—Lap joints are used for the butts and seams of all shell plates. The butts of the inner strake are joggled inboard and those of all outer strakes joggled outboard. Two assembly holes are provided on the seams at the intersection of frames; one at the ends of each butt and one on each seam at the intersection of each floor.

If plates are found not to be in contact at butts, seams, floors, and frames, a nut is tack-welded to the shell plating where necessary, and by the use of the jig shown in Fig. 51-*D* or by the tack-welding of additional clips to frames and floors (Fig. 52-*B*), it can be pulled up satisfactorily. This method eliminates the cost of punching and the welding up of the holes. Where the use of nuts is not practical, as may be found at the ends of the ship (Fig. 63), additional holes are punched in the plating with the torch, halving and quartering the center-to-center distances of all existing holes, as previously outlined.

Methods *B*, *C*, *E*, or *F* of Fig. 51 then may be used to pull the plates to the frames.

Floors (Fig. 60).—All partial floors are subassembled on the slab and welded without the use of assembly holes. Assembly clips are, however, bolted to the solid and partial floors to

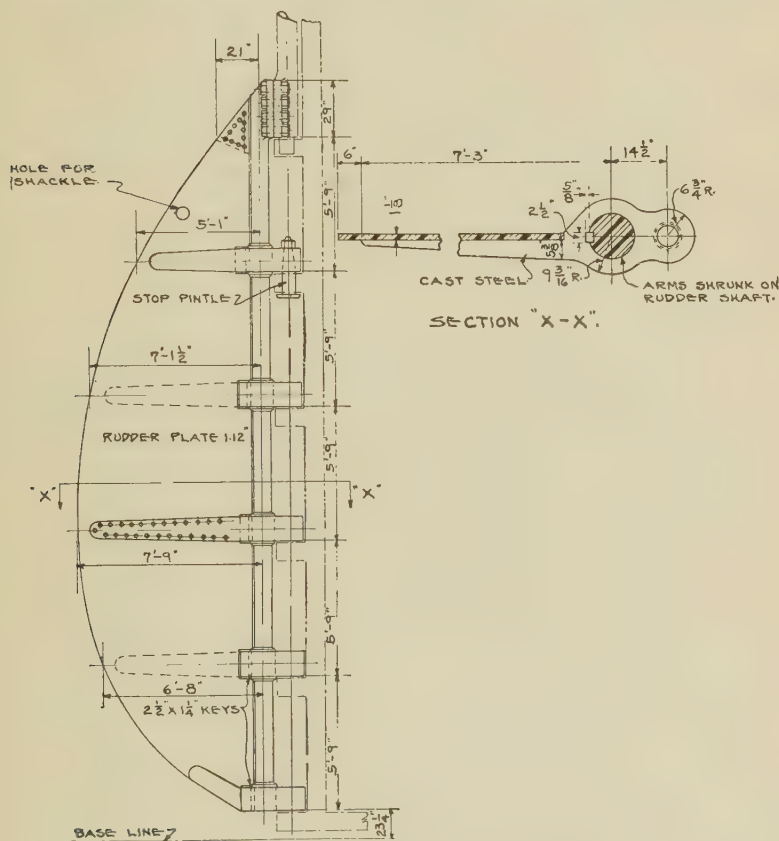


FIG. 64.—Riveted rudder for 13,220-ton dead-weight freighter.

correspond with the assembly holes in the shell and inner bottom, and holes are punched to take assembly clips on the keel. The outboard ends of all floors, except those at the extreme fore and after sections of the ship (Fig. 63), have an adjustable plate (Fig. 60) that is fitted on the ship, thereby eliminating the difficulty usually encountered in riveted construction of fitting the bilge strake to the floors.

Inner Bottom and Decks (Fig. 60). As the surfaces of the inner bottom and the decks are straight, and as standard angles have been used for the framing of partial floors and for deck beams, no advantage is seen in slotting the toe of the angle, as has been done on the frames; therefore the standard

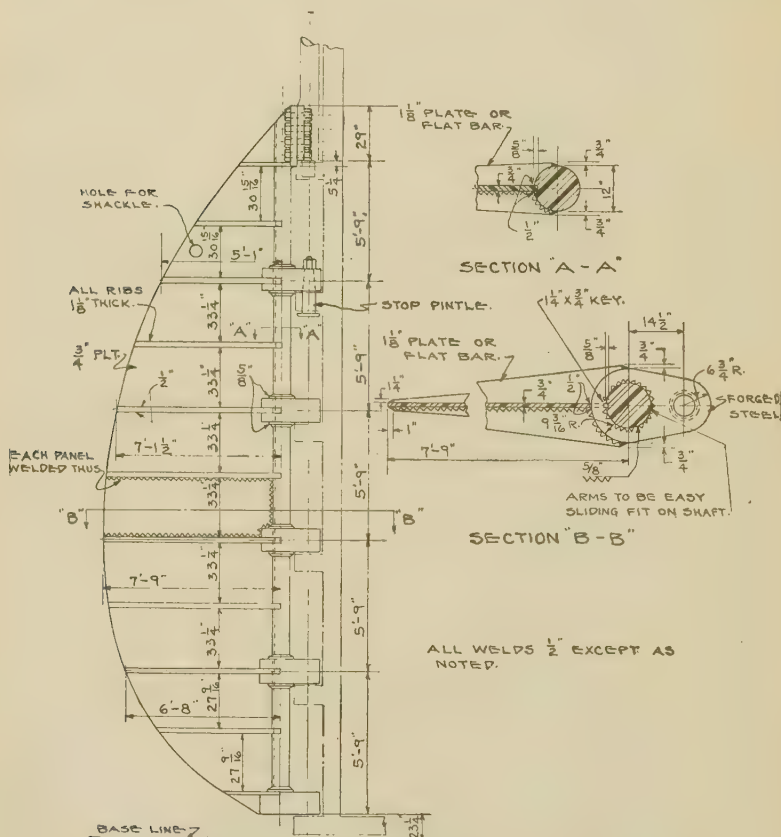


FIG. 65.—Welded rudder for 12,720-ton dead-weight freighter.
Weight saved, 7.9 per cent.

method of joggling butts and seams has been adhered to. The general method of assembly is similar to that of the shell plating.

Foundations.—All foundations are completely subassembled and extensively automatically welded on the welding slab without the use of assembly holes. Clips are provided for assembling foundations on the ship. Main engine foundations for riveted and welded construction are shown in Figs. 66 and 67.

Frames (Fig. 60, Details WJ-1 and 2).—The frames are cut with the torch on a production basis from standard I-beam sections, providing a bracket integral with the frame. This bracket is formed in a double-ram hydraulic press, the vertical ram serving as a clamp and the horizontal ram opening the beam. By means of bolted pieces the dies can be made adjust-

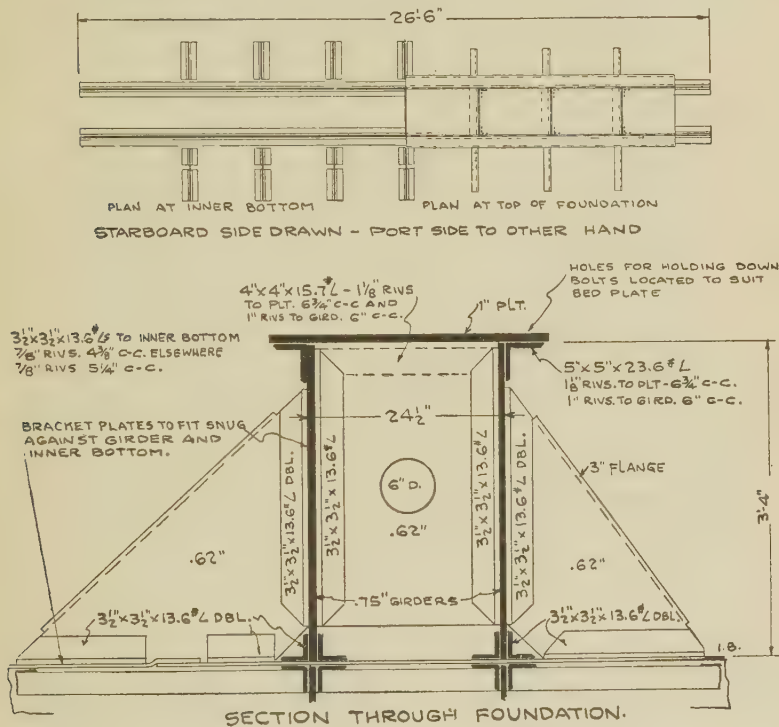


Fig. 66.—Riveted construction of main engine foundation.

able to any size of beam. In some cases these bends may be made cold, but if necessary the beam can be heated locally in an oil furnace. The web of the frame is notched out in the way of the inner strakes of the shell plating, thereby eliminating the joggling of the shell plating or frames, which has been the practice in the past. Slots are also provided in the flange in the way of beam brackets for the insertion of a bar. This bar is provided with two holes for the assembly of the deck beams. Clips are tack-welded on both sides of the web at every seam for pulling

the shell up to the frame. This frame construction makes possible the obtaining of port and starboard frames that are absolutely identical and eliminates the possibility of errors. The frame bracket, although of equivalent strength to the standard riveted bracket, is very much smaller and 71.4 per cent lighter. This small bracket will make possible the obtaining

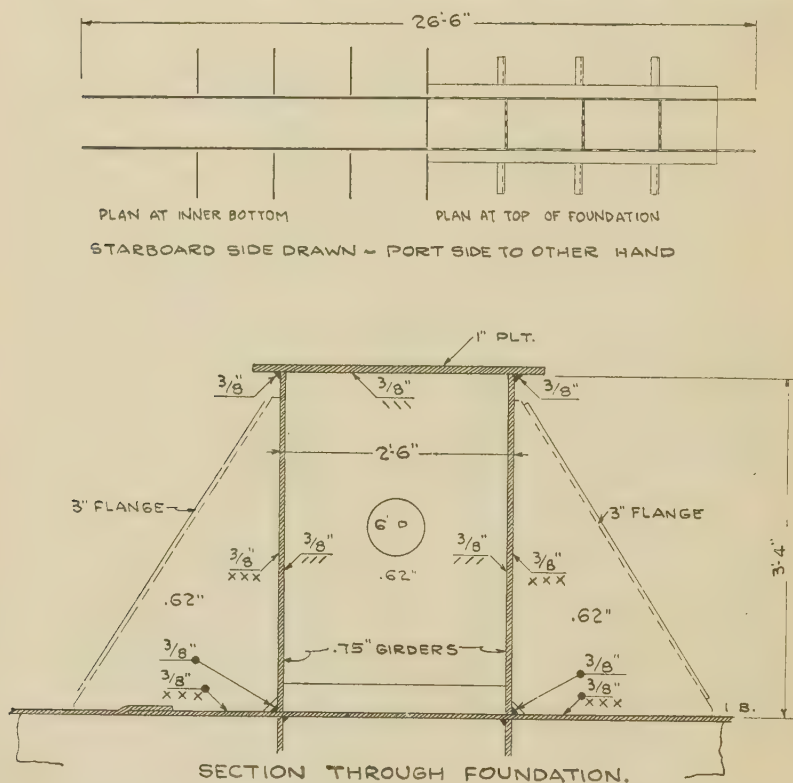


FIG. 67.—Welded construction of main engine foundation.

Saving in weight, 3.18 per cent.

of frames continuous with floors, if desired, the margin plate being continuous inboard of the frames. In this case intercostal plates, similar to those used on the decks, would be fitted between the frames.

In the parallel-middle body of the ship, where the shell plating lies at right angles to the frames, the square edge of the frame as cut by the torch is desirable, as shown in Fig. 53A. But

forward and aft, where the ship is beginning to fine, the plating will be at an angle to the frame, and one edge of the latter would stand away from the plating to such an extent as to require an excessive deposition of weld metal to secure an adequate connection. In the ship in question the maximum angle which the plating will form with a tee frame is about 20° . By milling the tee, as shown in Fig. 54, the frame can be made to lie close enough to the plating for all practical purposes. A 6° angle was chosen as representing a desirable mean.

Figure 54 shows a portable edge-milling machine designed to do this milling. The machine will be clamped to one end of a long frame and will feed itself automatically along the latter. The angularity of the feed roller will hold the cutter up to its work, and the guide rollers will cause the cutter to follow the contour of the joggles.

It will be necessary only to mill about half of the frames in the ship; the others need only to be smoothed off.

In the extreme ends of the ship the framing changes to angles. These are already rounded and will fit to the plating, as shown in Figs. 53, *E* and *F*. The extreme condition is shown in the latter, and there is only a very small amount of such angularity.

Stem and Stern Posts (Fig. 60, Detail WJ-11).—The stem and the stern posts are of standard construction, but provided with $1\frac{3}{16}$ in. diameter rivet holes spaced 6 in. center-to-center for the effective bolting up (prior to welding and riveting) of the shell plating and for the composite joint used to attach the shell plating to the stem and the stern frame.

Deck Stringers (Fig. 60, Detail WJ-2).—The weight of the stringers on all decks has been maintained the same as that of riveted construction. On the second and third decks an intercostal plate is provided between the frames for attachment of the stringer to the shell and frames, thereby avoiding the notching of the stringer plate, eliminating stringer and intercostal angles, and providing an effective watertight construction. Another important feature of this construction is that the intercostal plates when welded to the frames form a continuous stringer adjacent to the shell plating, thereby giving additional strength to the stringer.

Pillars (Fig. 60, Detail WJ-13).—Types of pillars and construction details are shown in the Appendix, Fig. 28. It is purposed to use Detail A, as it is considered that one $\frac{5}{8}$ -in.

continuous fillet weld around the pipe provides ample strength. The plate on the bottom of the pillar performs the function of the spring plate required in riveted construction and that on the top of the pillar is used as a butt strap for the bottom cover plate of the deck girders, which are spliced on the center-line of pillars.

Main Transverse Bulkheads (Fig. 62).—In general the construction and fabrication of the stiffeners and brackets of the main transverse bulkheads below the third deck are similar to the frames and brackets that previously have been described.

The bulkheads are plated with the seams transverse to the stiffeners, using joggled seams and butts.¹ The seams are joggled on the stiffener side, and the stiffener is notched with a torch in the way of a joggle, so as to provide a flush plating surface on the side away from the stiffener for dogging down to the slab. When an automatic machine, similar to that shown in Fig. 28, is available, it is purposed to use unstrapped butt joints instead of joggled laps. All bulkheads are provided with a deep bounding angle lapped on the plating. A lap joint is necessary for the attachment of this bounding angle to the bulkhead plating, to permit adjustment of overall dimensions of the bulkhead after completion of the welds in the plating and readily to fit the bulkheads to the contour of the ship. The deep angle is used to make possible the welding of the overlapping plate edge, either on the slab or on the ship.

To assemble and weld the bulkhead on the slab, 1-in. nuts are first tack-welded on 15-in. centers to the bulkhead plating on the side adjacent to the slab; the plating is then laid down on the slab and held down, as shown in Fig. 30.² The seams and butts are then welded by the automatic, after which the stiffeners are tack-welded in place and automatically welded to the plating. Bounding angles, if welded on the slab, are then lapped on the edges of the plating, the overall dimensions are

¹ Experience obtained subsequent to the preparation of this paper indicates that it would be preferable to have the seams and stiffeners run in the same direction so as to reduce warping to a minimum. The author's experience has been independently verified by Chief Constructor Lottman of the German Navy.

² Instead of bolting the bulkhead to a slab as outlined, the Newport News Shipbuilding and Dry Dock Company now tack welds a series of nuts to the bulkhead to secure one or more ribbands. The ribbands tend to prevent excessive warpage and are necessary for handling.

carefully checked, and the edge is automatically welded. The bulkhead is now turned over, and all welds on the other side are completed with the automatic or subsequently by hand on the ship. It will be noticed that the fillet welds of seams, if welded manually on the ship, can be made in the "flat" position, as the laps are designed to permit this. The top brackets of stiffeners are completely shop-welded, whereas the bar that attaches the bottom bracket to the inner bottom is welded on the ship after the bulkhead is assembled, thus making possible adjustment for slight inaccuracies in deck heights. The boundary angle is provided with assembly holes for location purposes. It should be noticed that the provision for "adjustment" is just as necessary in the riveted ship as in a welded ship and is customary in all steel shipbuilding.

Miscellaneous Bulkheads (Appendix, Fig. 29).—The stiffeners of miscellaneous bulkheads and strength bulkheads above the third deck (Fig. 62, Detail WJ-7) are in general of angle construction without top and bottom brackets, the toe of the angle being welded to the plating. In other respects their method of construction, assembly, and welding is similar to the transverse strength bulkheads previously described.

Casings.—In general casings are similar in design to the bulkheads, and the same methods of assembly and welding are followed. No reduction in the weight of plating is assumed, although it may be found practical.

Girders and Cargo Hatches (Fig. 55 and Details WJ-8 to 13 of Fig. 60).—Welded girders are of simple construction, being built up of a continuous web, a bottom cover plate, and a deck plate that forms the top cover plate. The usual continuous channels, intercostal angles, deck clips, and bracket clips used in riveted construction are eliminated. The top cover plate of the girder is extended beyond the corner of the hatches so as to form a double plate (Detail WJ-12). On the upper deck the web of the girder, which forms the fore and aft coaming of the hatches, is carried past the ends of the hatches. One of the principal features of design shown is that the web plate is not cut to permit the use of continuous deck beams, all beams being intercostal and securely attached to the girder, as shown in Detail WJ-10. The method of attachment used insures ample strength and continuity of deck beams as in riveted construction, but necessitates a change in the usual order of erection, for the

pillars and girders will have to be installed prior to the deck beams.

The length of deck girders throughout the ship is that of the pillar centers. They are spliced on the web by a double butt strap; on the bottom cover plate by using the capital plate of the pillar as a butt strap; and the shift of the butts of the deck plating insures that the butt of the top cover plate is in another frame space. The butt of the top cover plate is the standard joggled lap used for all deck plating.

Owing to the sheer of the ship, it will be necessary to cut the webs of these girders with a torch, the girders and all tripping brackets being automatically welded on the bending slab in a manner similar to the keel. It will be necessary to use extreme care to be assured of the proper sheer when welding is complete. The two assembly holes shown in the tripping brackets are provided to facilitate erection of deck beams, one hole being used for a drift pin to permit insertion of a bolt in the second hole.

Companion Hatches.—The coamings of companion hatches are constructed similarly to those for manholes, as shown in Appendix, Fig. 31, and the covers for these hatches are similar to that shown in Appendix, Fig. 32. To be assured of accurate cutting of openings in the decks, it would be necessary to lay off these openings directly from the coamings and cut with the torch.

Hatch Covers.—The main cargo hatch covers are of wood supported by steel hatch web beams, as shown in Appendix, Fig. 33. The wooden hatch covers are customary on most riveted ships. The hatch beam socket is shown in Appendix, Fig. 34, and preferably is shop-welded. Main cargo hatch covers may be of metal, as shown in Appendix, Fig. 32. The hinges and dogs are of a simple bar construction replacing the expensive cast-steel and drop-forged designs now in general use. The main features of design are shown in Appendix, Fig. 35, the rubber gasket being mounted in a rolled channel welded to the cover.

✓ **Deck Beams** (Fig. 60).—All deck beams are of angle construction with the toe of the angle adjacent to the deck plating. The ends connecting to the frames are split with the torch and opened up to form a bracket similar to that used on the frames and bulkhead stiffeners. All deck beams are intercostal between

deck girders and shell. The method of assembling and connecting these beams to the girders and frames and the deck plating to the beams has been described previously.

Masts.—The masts of this ship are similar in design to those proposed for the S.S. "Virginia" and as shown in Fig. 48. The principal features of the design are that the width of the butt straps is very much reduced and the doublers are in narrow widths to obtain edge connections in place of the usual stitch riveting. All longitudinal seams preferably should be automatically welded on the slab after assembly of the mast. Mast rings have been eliminated and the mast tables very much simplified.

Mast and Boom Fittings.—It will be noted from the drawing of the mast that all fittings are completely welded, no cast or drop-forged fittings being used throughout the design. It may be objected that this type of fitting is difficult to repair; however, this is not considered to be a pertinent objection, as worn parts can be built up readily with the arc, and in case of broken parts readily removed with a gas torch and replaced at a lower cost than would be the case for the usual fittings. Where this type of fitting replaces the usual mast band, it has been carried diametrically through the mast wherever practical, thereby strengthening the mast.

Rudder.—Figure 64 shows the standard design of a single-riveted plate rudder, and Fig. 65 the corresponding welded rudder. The design of the latter is very similar to the former, except that the arms are now made as shown in the drawing. Owing to the fact that intermediate arms have been used, it has been possible to decrease the plate thickness from $1\frac{1}{8}$ to $\frac{3}{4}$ in. It no longer is necessary to shrink the arms onto the shaft, as welds are used to secure them to the latter. A continuous keyway has been cut in the shaft and the plate inserted in this slot, thereby insuring accurate centering and facilitating assembly. Owing to the large amount of welding on this rudder and the experience with previous rudders, it has been found that very thorough dogging is necessary to prevent warping and that all welding should be completed before the final machine work.

Smokestack.—To date it has not been considered economically advantageous completely to weld smokestacks and uptakes, because of the light construction and widely spaced rivets.

However, improved welding production methods in the future might make this possible. Figure 49 in the Appendix shows a completely welded stack for a destroyer. A stack for a merchant ship with a welded spider and other features is shown in Appendix, Fig. 50.

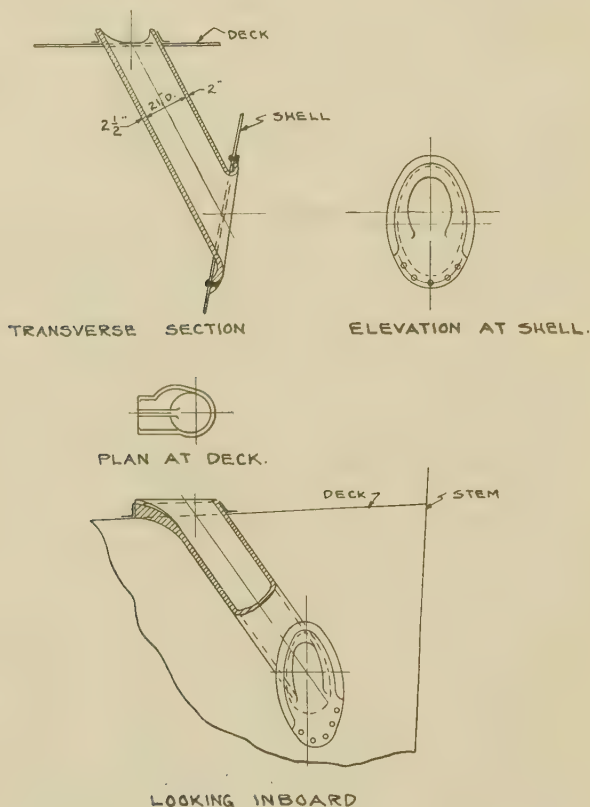


FIG. 68.—Cast-iron hawse pipe for 12,720-ton dead-weight freighter.

Hawse Pipe.—Figure 68 shows the usual cast-iron hawse pipe and Fig. 69 a corresponding welded one. In the manufacture of a cast hawse pipe a large portion of the bow of the ship, together with the deck, is built up in the pattern shop to full scale in order to make the necessary patterns. Two complete patterns, one right and one left, are required. After the pipe has been cast, it is seldom that it accurately fits the ship, and liners and other methods must be resorted to so as to fit it in

place. By substituting a welded pipe, however, all this expensive pattern work is eliminated. Such molds as may be necessary can be taken directly from the ship.

In erecting the pipe in the ship, if the former does not fit exactly, adjustments can be made then and there by heating. After the pipe proper has been installed, the ribs shown at the

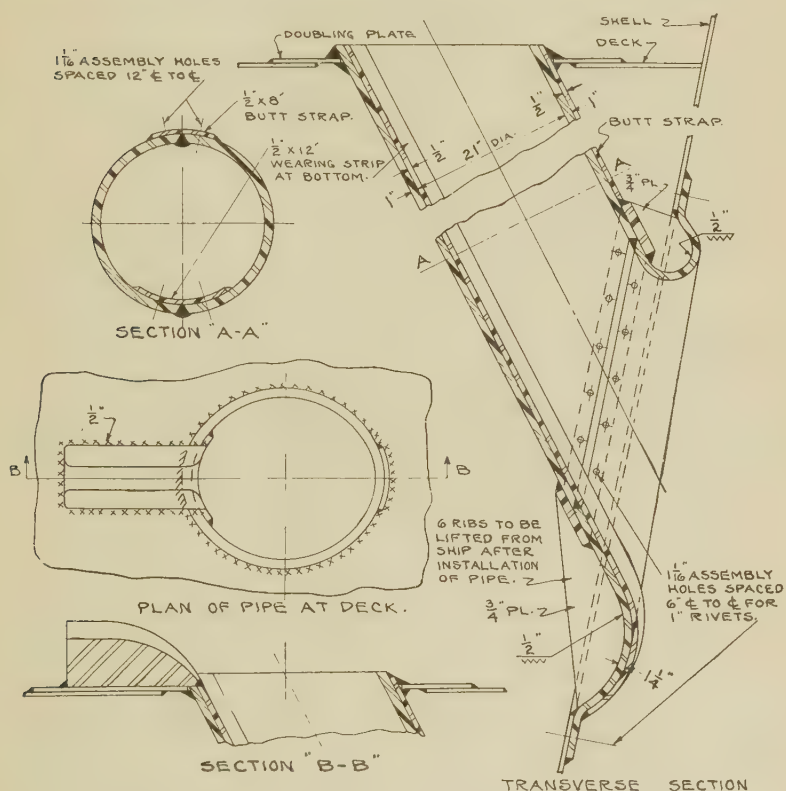


FIG. 69.—Welded hawse pipe for 13,220-ton dead-weight freighter.

shell can be welded in place. If desired, the wearing strip shown could be welded on the bottom side of the pipe, either at the making of the pipe or later on as a repair. It will be noted that rivets are used to attach the pipe to the shell. This was done because of the necessity for having many large bolts to draw the pipe to the shell.

Vents.—The general construction of the vent trunks used in this ship is shown in Appendix, Figs. 37 to 43. Some slight modifications will be necessary, as the tentative standards for these welded vent trunks were specifically prepared for riveted ships. It is purposed to keep the weight of the plating used in the trunks the same as has been considered good practice for riveted ships. The principal advantage of welded vents is the reduction in cost by the maximum elimination of flanged connections, the use of butt joints, and narrow laps. It is purposed to use automatic machines extensively. A decided advantage of welded construction is the insuring of water-tightness throughout the vent system, as riveted vent coamings have always given much trouble because of leaks from water that has collected in inaccessible places.

Miscellaneous Parts (Water Sheds, Rail and Awning Stanchions, Etc.).—Arc welding will be extensively applied to the welding of miscellaneous parts of the ship such as water sheds and rail and awning stanchions. Such parts, however, will not be considered in detail.

Welding of Machinery.—The condensers, independent tanks, fuel-oil-heating coils, floor-plate supports, ladders and gratings, hull fittings, and to some extent the hull piping, will be welded.

CHAPTER VI

RELATIVE WEIGHT, COST, AND ECONOMY OF RIVETED AND WELDED SHIPS

General.—The cost of constructing riveted or welded freighters of the size and type described in Chap. V will depend upon the price of materials and labor at the time of construction, the country in which the ship is built, and the amount of work on hand in the shipyard. This last factor has a marked effect on the overhead expense, which, together with the other factors, makes it impracticable to tie down the costs to figures that could not be questioned.

In view of the foregoing, a method permitting interpolation will be used, so that the reader may obtain comparisons suited to his particular building conditions. This method assumes that the cost per deadweight ton¹ for constructing the riveted freighter would lie between \$50 and \$100, regardless of where built. This basis of cost determination also will be used later in arriving at the economic advantage of welded construction as compared with riveted construction.

Saving in Weight.—In the detailed estimate of the comparative weight and carrying capacity of the riveted and welded freighters given in Table 4, it will be noticed that the hull steel of the welded freighter is 494 tons (2,944 – 2,450) less than that of the riveted freighter, or a saving of 16.7 per cent. It also will be noticed that at a 31-ft. 8½-in. draft the deadweight capacity of the riveted ship is 12,719 tons and that of the welded ship 13,219 tons, or an increase for the latter of 500 tons, or 3.9 per cent. The drafts given for these freighters are 31 ft. 8½ in. and 28 ft. 0 in. The former is the limiting draft for which the vessel is designed, and the latter is, for general cargo-carrying work, about as deep as many harbors will permit. With a 28 ft. 0 in. draft the welded ship will carry 4.75 per cent more deadweight than the riveted ship.

¹ Deadweight tonnage is the measure of the weight-carrying capacity of ships, and it must not be confused with gross tonnage, which is the measure of volume and which will be used later in the discussion.

TABLE 4.—COMPARATIVE WEIGHT AND CARRYING CAPACITY OF THE RIVETED AND WELDED FREIGHTERS DESCRIBED IN CHAP. V

Item	Weight in long. tons (2,240 lb.)	
	Riveted	Welded
Net floating steel.....	2,944	2,450
Wood and outfit.....	499	499
Hull engineering.....	172	172
Machinery (wet boilers).....	736	730
Margin.....	100	100
Light weight.....	4,451	3,951
Displacement, 31 ft. 8½ in. draft.....	17,170	17,170
Deadweight, 31 ft. 8½ in. draft.....	12,719	13,219
Displacement, 28 ft. 0 in. draft.....	14,990	14,990
Deadweight, 28 ft. 0 in. draft.....	10,539	11,039

Saving in Initial Cost.—An estimate shows that the approximate cost of material for the welded ship will be 6.1 per cent less than that of the riveted ship and that the labor will be about 5 per cent less. The total cost of the welded ship will be approximately about 3.7 per cent less than the riveted ship. This total cost includes labor, material, overhead, and a small profit. The overhead expense has been considered the same for both the riveted and the welded ships.

On a basis of this 3.7 per cent saving in the cost of the welded ship there is obtained the comparative initial cost of the riveted and welded ships given in Table 5.

TABLE 5.—COMPARATIVE INITIAL COST OF THE RIVETED AND WELDED FREIGHTERS DESCRIBED IN CHAP. V

Cost per ton deadweight (riveted)	\$50	\$100
Cost of riveted freighter.....	\$635,950	\$1,271,900
Cost of welded freighter.....	612,420	1,224,840

Saving in Cost of Operation for the Welded Freighter.—In order to establish a basis for the savings in the cost of operation that will be secured when ships are of welded construction, the

savings for the riveted and welded designs will be considered. Table 6 is an analysis of the comparative carrying capacities;

TABLE 6.—COMPARATIVE CARRYING CAPACITY FOR A VOYAGE OF 4,000 MILES MADE BY THE RIVETED AND WELDED FREIGHTERS DESCRIBED IN CHAP. V

Item	Weight in long tons (2,240 lb.)	
	Riveted	Welded
Fresh water.....	60	60
Feedwater.....	100	100
Stores.....	30	30
Equipment and sundries.....	75	75
Coal consumed at sea.....	820	820
Coal (reserve).....	150	150
Coal consumed in port.....	50	60
Deadweight (cargo).....	9,254	9,744
Total deadweight (28 ft. 0 in. draft)....	10,539	11,039

Table 7, an estimate of the number of one-way trips per year that could be made; and Table 8, the annual cost of operation

TABLE 7.—ESTIMATE OF NUMBER OF AVERAGE 4,000 MILE ONE-WAY TRIPS PER YEAR COULD BE MADE BY THE RIVETED AND WELDED FREIGHTERS DESCRIBED IN CHAP. V

Operation	Time in days
Loading cargo.....	5
Loading bunkers.....	1
En route.....	14½
Discharging cargo.....	5
Lost time.....	½
Total time.....	26
Number of one-way trips per year allowing 27 days for docking, overhauling, and for layovers.....	13

for the riveted and welded freighters. The data given in Table 6 are based on a typical 4,000-mile voyage, with general cargo, making $11\frac{1}{2}$ knots average sea speed, with 3,500 i.hp. The draft has been taken at 28 ft. 0 in. for the reason previously explained.

These data show that the all-welded ship can charge 5 to 6 per cent less for carrying cargo and yet pay the same percentage

in dividends as the riveted ship. This means that when all ships are welded the freight rates would drop about 5 to 6 per cent, assuming that other things are equal. On the other hand, if the present freight rates are held to, which will probably be the case until a considerable proportion of the world's ships are welded, the welded-ship owner will have this added earning capacity for increased dividends.

TABLE 8.—ANNUAL COST OF OPERATION FOR THE RIVETED AND WELDED FREIGHTERS DESCRIBED IN CHAP. V

Item	Riveted		Welded	
Depreciation at 5 per cent.....	\$ 31,797	\$ 63,595	\$ 30,621	\$ 61,242
Insurance at 8.5 per cent.....	54,000	108,000	52,000	104,000
Office charges.....	14,550	14,550	14,650	14,650
Repairs and docking ¹	5,620	5,620	5,620	5,620
Stores ¹	5,620	5,620	5,620	5,620
Wages and food ¹	45,000	45,000	45,000	45,000
Harbor fees (approximate).....	16,000	16,000	16,000	16,000
Stevedoring ¹	60,200	60,200	63,350	63,350
Coal ¹	56,550	56,550	57,200	57,200
Assumed dividend at 5 per cent....	31,797	63,595	30,621	61,242
Total expenses.....	\$321,134	\$438,730	\$320,682	\$433,924
Cargo carried per year, long tons....	120,400	120,400	126,700	126,700
Corresponding freight rate per ton..	\$2.67	\$3.64	\$2.53	\$3.43

¹ The cost of these items is based on average prices, as the initial cost of a cargo ship may vary, depending upon where it is built, but the operation is international. Wages and food are a compromise between European and American rates.

Increase in Earning Capacity.—Table 9 shows the comparative earning capacity of the riveted and welded freighters. An increase in dividends of approximately 50 per cent by a method of construction that requires less capital investment must be of interest to ship owners. Similarly, if rates are low and the owner of a riveted ship is unable to declare a dividend, the owner of a welded ship has an advantage that may mean the difference between failure and success.

Savings That Could Be Effected if All Ships Were Welded.—Let consideration now be given to the effect of having all of the world's steel ships welded instead of riveted. The variety in size and type of ships, length of voyages, prevailing freight

TABLE 9.—COMPARATIVE EARNING CAPACITY OF THE RIVETED AND WELDED FREIGHTERS DESCRIBED IN CHAP. V

Item	Riveted		Welded	
Capital investment in ship.	\$635,950	\$1,271,900	\$612,420	\$1,224,840
Dividend (5 per cent) previously allowed for.	31,797	63,595	30,621	61,242
Additional earning capacity for one year at same freight rate.			17,738	26,607
Total net earnings.	31,797	63,595	48,359	87,849
Corresponding dividend, per cent	5	5	7.9	7.2

rates, etc., make it impracticable to give a figure for probable savings in the cost of operation that could not be questioned. This analysis is, however, believed to be a fair method of comparison and will at least show that a tremendous amount of money will be saved by welded ships.

As previously explained, some 80 per cent or more of the world's tonnage is in freight carriers, and the riveted and welded vessels that have been discussed are typical of the modern, moderately fast freighter. If, therefore, it is assumed that the savings effected by the use of welding will be typical of 80 per cent of the world's future steel ships, the saving to the world on all water-borne freight can be approximated.

In 1927 there are 61,303,700 gross tons of steel ships in the world of over 100 gross tons, not including sailing vessels. Of these, approximately 56,394,789 gross tons are vessels of over 1,000 gross tons and accordingly are sizable vessels suitable for use in this comparison. In view of this, it may safely be considered that 45,000,000 gross tons are freight vessels.

As a welded freighter of 8,100 gross tons saves from \$17,738 (on \$50-a-ton cost basis) to \$26,607 (on \$100-a-ton cost basis) per year, then the average welded freighter of 8,100 gross tons should save about \$20,000 a year, or \$2.47 per gross ton.

On this basis ship owners and operators would be able to save \$111,150,000 (45,000,000 times \$2.47 per ton) per year on 45,000,000 gross tons of water-borne commerce. On the other hand, when a considerable proportion of the world's tonnage is welded, freight rates will be adjusted to the changed conditions and society as a whole will benefit. Until this takes place, the owner of welded ships will have a distinct advantage because of the greater earning capacity of his ships.

CHAPTER VII

EFFECT OF THE FULL UTILIZATION OF ARC WELDING ON SHIPYARDS AND ON INDUSTRY IN THE UNITED STATES

General.—In a previous chapter it has been shown that the cost of ships can be reduced 3.7 per cent by the use of welding. This comparison was made on the basis of the same overhead applying to both riveted and welded ships and on a plant layout and equipment designed for riveting. But certain economies will be effected in the operation of shipyards owing to the complete adoption of welding, and it is purposed to show that these same economies will apply not only to shipyards but to any industry using steel as its raw material and to show their effect on industry as a whole in the United States.

In most plants at the present time only those items are welded that cannot be riveted economically. The analysis following will reverse this condition and attempt to show the savings that would be obtained when only those items are riveted that cannot be welded.

Effect of Arc Welding on Shipyards.—In Tables 10 and 11 will be found the estimated savings in material and labor that will be obtained when arc welding is fully utilized in shipyards. These percentages are based upon conclusions reached from the actual costs of a number of welded products, yet owing to the number of variables involved they must of necessity be subject to a variation of several points of percentage. These figures have been prepared on the basis of a shipyard doing a \$20,000,000 business annually and in which more than 30 different trades are employed. The direct materials are affected in quantities easily determined from designs, but large savings also will be effected in the indirect materials, savings that are not easy to determine.

To illustrate the *influence of welding on indirect materials* when a full utilization of welding is secured, consider the cost of compressing, delivering, and utilizing air, which cost in a

TABLE 10.—ESTIMATED SAVINGS IN MATERIAL IN SHIPYARDS WHEN ARC WELDING IS FULLY UTILIZED

	Percentage		
	Decrease on item	Increase on item	Increase or decrease on total direct material
Direct material:			
Steel plates, shapes, bars, etc.....	15	5 ¹	10.55
Oxygen and acetylene.....	..	20	0.18
Castings, iron.....	25	...	1.06
Castings, steel.....	25	...	0.32
Electrode material.....	..	100	0.12
Items not affected.....			
Net saving in per cent.....	..	...	11.63
Indirect material:			
Assembly bolts.....	25	...	1.28
Rivet heating fuel (coke and oil)...	75	...	2.5
Air for riveting, drilling, calking...	50	...	11.1
Air hose.....	75	...	1.5
Purchased power.....	..	10	2.23
Items not affected.....			
Net saving in per cent.....	..	...	14.15

¹ Increase due to steel required for castings.

large shipyard is about 25 per cent of the total cost of indirect materials. In this connection it has been stated that air is only about 10 per cent as efficient as the same power delivered electrically, owing to losses in generation, distribution, utilization, thoughtless applications, etc. Regardless of future improvements in air-compressing and air-using machinery, the great losses of leakage and wastage still will exist.

When welding is fully utilized in such a shipyard, at least 75 per cent of the air hammers and drills now required would be eliminated and the capacity of the compressing plant itself could be reduced 75 per cent. As the distribution lines, totaling approximately three miles, could not be reduced, the saving in the cost of air has been reduced only 50 per cent instead of 75 per cent.

Very efficient electric drills now can be obtained, but electric hammers have not yet reached as high a degree of perfection.

TABLE 11.—ESTIMATED SAVINGS IN LABOR IN SHIPYARDS WHEN ARC WELDING IS FULLY UTILIZED

Department	Percentage		
	Decrease on item	Increase on item	Decrease or increase on total labor
Ship fitters.....	15	...	0.77
Drillers.....	75	...	1.09
Erectors.....	10	...	0.11
Mold loft.....	30	...	0.27
Riveters.....	75	...	4.19
Anglesmiths.....	15	...	0.19
Ship sheds (steel shops).....	25	...	0.78
Hull fitting shop.....	10	...	0.58
Transportation.....	10	...	0.21
Plumbers and pipe fitters.....	10	...	0.24
Sheet-metal shop.....	5	...	0.07
Arc welders.....	..	100	1.09
Boiler and tank shop.....	10	...	0.17
Main machine shop.....	5	...	0.19
Pattern makers.....	50	...	0.51
Drafting rooms.....	15	...	0.72
Gas cutters.....	..	20	0.15
Other departments not affected.....			
Supervisory and clerical.....			
Net saving in per cent.....	..	...	8.85

If a successful electric hammer is developed, the use of air could be practically eliminated; on the other hand, there will be little use for a hammer except for chipping and peening in a yard in which welding was fully developed.

Bull riveting is now extensively used to reduce cost and improve riveting, but automatic welding performs the same function for welding that the former does for riveting; that is, it produces a superior product at a lower cost and is rapidly extending its field of usefulness. Further, automatic welding heads are light, portable machines and can be used where it is not possible to apply cumbersome bull riveters.

In the driving of watertight rivets a gang will have with them one pneumatic hammer for the holder-on, one and often two for the riveter, one for the calker, one drilling machine for the reamer, and one air-draft oil or coke furnace. The total cost

of this equipment is at least \$700 as compared with approximately \$350 for a portable resistor and reactor for use with a constant-potential motor-generator set, as shown in Figs. 24 and 25. The cost of the generating equipment and leads is much less than the corresponding cost of air-generating equipment and hose, and the latter is more subject to wear and depreciation. In view of the foregoing, serious consideration should be given in the construction of new plants to the full utilization of arc welding and the other welding processes, because of the smaller investment required and the rapid growth of the process, which already is making radical changes necessary in plant equipment and layout.

A completely modernized plant should be laid out to use the minimum amount of air, and it should have compressing, distributing, and utilizing equipment sufficient for supplying air to only the few hammers necessary for the small amount of chipping and peening that will be required. All furnaces should be provided with independent blowers, and all portable drilling machines should be of the electric type. The welding installation should be laid out to utilize the most efficient and economical generating, distributing, and utilizing equipment to suit the conditions. Automatic welding and gas cutting machines (rectilinear and rotary), ample welding slabs, and various types of assembly devices should be provided.

In shipyards, as in many other plants, there will be a number of trades that will not benefit by welding, such as joiners and painters. Similarly, the materials used by these trades, such as paint and lumber, will not be affected. This being the case, the saving by the use of welding will be about 9 per cent for the entire plant and will amount to approximately \$1,900,000 for the size of yard considered in this analysis. When completely welded ships are built in such a yard, the savings effected by welded construction will be much more than the 3.7 per cent used in estimating the cost of the welded ship described in Chap. V.

Effect of Arc Welding on Industry in the United States.—To show the savings that would accrue in industry throughout the country as a whole when welding is fully utilized is not as simple as a determination of the probable savings in a plant or an industry. In this analysis the writer has used statistics principally of the Steel Institute of America and of the Census

Bureau of the Department of Commerce for the year 1925, and probable savings will be considered from the standpoint of "saving in material" and the savings in "value added by manufacture"¹ (Tables 12 and 13).

The *probable saving in material* (Table 12) can be arrived at best by considering the saving in rolled steel and castings, because

TABLE 12.—ESTIMATED SAVINGS IN ROLLED STEEL AND CASTINGS IN THE UNITED STATES IN 1925 IF WELDING HAD BEEN UTILIZED FULLY

Product	Tonnage, long tons	Value of product, dollars	Saving, per cent	Value of saving, dollars
ROLLED STEEL				
Rails.....	2,703,520	114,334,375		
Rails (rerolled or renewed).....	70,900	2,652,161		
Rail joints, tie plates, etc.....	818,096	46,723,816		
Concrete bars.....	646,711	32,284,085		
Wire rods.....	2,999,551	54,713,566	0.24 (Increase)	130,000 (Increase)
Spike and chain rods.....	177,069	2,498,619		
Bolt and nut rods and rivet rods....	203,233	813,124	35	280,000
Sheets (No. 13 and thinner).....	5,607,791	234,437,676		
Skelp.....	3,445,149	54,430,784		
Ties, bands, and strips.....	1,733,043	86,048,304		
Axles and wheels.....	282,735	26,731,413		
Armor.....	1,071	424,208		
Products not listed elsewhere.....	1,025,410	68,605,991		
Nail and tack plate.....	31,621	801,715		
Plates, shapes, and bars.....	13,061,789	653,982,566	2.6	17,000,000 ¹
Total.....	32,807,689	1,379,482,403	1.24	17,150,000
FERROUS CASTINGS				
Steel castings.....	1,007,913	144,414,553	3	4,300,000
Iron castings, gray and malleable....	4,586,097	428,121,974	30	130,000,000
Net saving in rolled steel and castings.....			7.8	151,450,000

¹ This figure is the difference between a \$63,000,000 increase due to steel for castings and a 12.2 per cent decrease (\$80,000,000) in plates, shapes, and bars.

these items are the principal ones affected by arc welding. By considering the material at its source, the possibility of duplication is eliminated.

Savings on bolt, nut, and rivet-rod steel will be effected through the large reduction in the use of rivet rods. Plates, shapes, and

¹ "Value added by manufacture" includes labor, overhead, and profit and is a term used by the Department of Commerce to express the difference between the "value of the product" and the cost of material used in the manufacture of the product.

bars have been grouped as it is difficult to determine the individual savings. The use of wire rods will be increased owing to the increase in welding wire. Steel castings as a whole will be affected but slightly owing to the fact that there is a large tonnage, such as freight-car castings, to which welding probably cannot be applied. Iron castings, on the other hand, will be materially reduced owing to the fact that a stronger, better, lighter, and cheaper product can be obtained by welded steel. In quantity production pressed steel is rapidly supplanting many castings, and welding is the ideal method of joining pressed steel parts.

The *probable savings in the "value added by manufacture"* (Table 13) is based on the principal manufactured products that

TABLE 13.—ESTIMATED SAVINGS IN THE VALUE ADDED BY MANUFACTURE ON VARIOUS MANUFACTURED PRODUCTS IN THE UNITED STATES IN 1925 IF WELDING HAD BEEN UTILIZED FULLY

Product	Value of product	Value added by manufacture		Saving	
	Dollars	Dollars	Per cent	Dollars	Per cent
Agricultural machinery.....	169,467,966	95,617,906	56	4,770,000	5
Cars (freight and passenger)...	390,771,210	124,717,706	32	2,500,000	2
Electrical machinery and supplies.....	1,540,002,041	903,309,965	59	45,000,000	5
Ornamental iron work.....	420,997,579	183,344,968	44	11,000,000	6
Shipbuilding, steel.....	132,415,163	82,857,361	63	2,500,000	3
Locomotives.....	65,389,134	23,671,992	36	710,000	3
Metal-working machinery.....	175,592,488	121,068,126	69	6,070,000	3
Engines and water wheels.....	313,587,851	167,804,164	53	8,400,000	3
Aircraft and parts.....	12,524,719	9,654,752	77	770,000	5
Laundry machinery.....	69,568,000	36,820,000	53	2,940,000	8
Automobiles (pleasure).....	2,483,885,306	850,000,000 ¹	34	8,500,000	1
Autos (trucks and tractors)...	3,198,122,633	1,089,930,821	34	3,260,000	3
Machines not listed elsewhere.	2,232,985,974	1,349,277,739	60	54,000,000	4
Structural steel.....	240,000,000 ¹	135,000,000 ¹	56	13,500,000	10
Total.....	11,445,310,064	5,173,075,500	45	163,920,000	3.15

¹ Estimated.

will be affected by welding. While there are a number of products not listed that will be affected slightly, the probable savings in these items doubtless will be offset by inaccuracies in apportioning the savings to those listed. Since steel is the only material that will be affected by welding and as the probable economies effected in steel have been taken care of already in

Table 12, each of the three elements of labor, overhead, and profit in the "value added by manufacture" will be influenced by labor.

In certain products, such as freight cars and automobiles, where the value of the material represents more than 50 per cent of the value added by manufacture, a saving in material is more important than a saving in labor. As such items usually are the result of intensive production, welding will make only a small reduction in labor on this class of work. Aircraft manufacture, on the other hand, represents the other extreme, wherein the value added by manufacture is many times the value of the material. An enormous increase and intensive production in aircraft construction will in the future reduce materially the cost of labor on this class of work, so that it probably will reach a level not far from that of other products. The average of the value added by manufacture for the products listed in Table 13 is only 45 per cent of the cost of the product, which means that *labor is in general not as important in cost reduction as is material*. It will further be noted that the percentage reductions made in the value of the product owing to the influence of welding are very conservative. It is probable that in actual production these values will be exceeded.

The saving in material for the United States in 1925 is estimated at \$164,000,000 (Table 12) and the saving in the value added by manufacture at \$150,000,000 (Table 13), or a total of \$314,000,000. As it will be a number of years before welding is adopted fully, an attempt has been made to arrive at the probable savings that will be effected by the process in 1940 by plotting a production curve of the various manufactured steel products for the last 27 years and extending it to 1940. It is found that by that time the production of steel in the United States, and therefore the material saved, will have increased 40 per cent over the figures given. As steel is the index of the activity of plants using steel as their raw material, this saving also will apply to the value added by manufacture, so that by the year 1940 a saving of some \$450,000,000 per annum can be expected over what would have been the cost of the products made by the methods of manufacture that would have been used had welding not been fully employed.

CHAPTER VIII

SUMMARY AND CONCLUSIONS

1. Arc welding can be applied to a wide range of work, from the small and light airplane joint to strength joints in the largest and most complex structure made of steel, a ship's hull. Practically all ferrous and non-ferrous metals can be welded satisfactorily by at least one of the arc-welding processes. Atomic welding, a combined arc- and gas-welding process and the latest welding development, is suitable particularly for the welding of the alloy steels and the non-ferrous metals.

2. Arc welding has reached a state of scientific and practical development which insures that reliable results can be obtained consistently. In support of this a large amount of evidence has been presented. For example, it is known why welded joints have failed in the past and how failure can be prevented in the future; what joints to use and when they should be used; that an entire arc-welding force of an industrial plant, including its apprentice welders (a total of 51 men), can secure minimum, maximum, and average test results, respectively, of 45,776, 71,648, and 60,922 lb. per square inch (Table 3); and that arc-welded structures have been subjected to and successfully have passed tests more severe than any imposed on a riveted structure, the most notable of which is a pipe line in which 72,600 tons of steel were used (Chap. IV).

3. Arc-welding possesses many practical advantages over riveting, the most important of which are higher joint efficiencies, tightness, savings in weight, and elimination of noise (Chap. I). Examples of joints having very high efficiencies in tension, shear, bending, compression, shock, and alternating stress have been given, also of joints that were water and oil tight under severe test conditions. Savings in weight were shown to be as high as 71.4 per cent (Fig. 56).

4. Arc welding possesses greater possibilities for reducing the cost and increasing profits by its use in new construction and for salvage than any known process or method (Chaps. I, IV, VI,

and VII). These savings are not confined to the value of the material or part saved, but they extend to labor and they influence overhead. In a shipyard doing a \$20,000,000 business annually, savings would amount to \$1,900,000 if welding were utilized fully. If all the freight ships of the world of over 1,000 gross tons were welded, \$111,150,000 would be saved annually by the owners and operators; or to put it another way, this saving would make possible a 4 per cent reduction in freight rates, thus benefiting society as a whole. The savings in material and the value added by manufacture for products manufactured in the United States would have been \$314,000,000 in 1925 if welding had been developed fully, and will probably be \$450,000,000 in 1940. It has not been possible to obtain data for the world similar to the data used in estimating the savings in manufactured products for the United States, but if it is assumed that the value of manufactured products of the world is the same as that of the United States, and that all ships and other mobile structures were welded, the savings annually, with the present volume of world business, would amount to over one billion dollars, and in 1940 to about one and a half billion dollars.

5. When arc welding has demonstrated more fully its reliability, the Ship Classification Societies undoubtedly will permit further reductions in the scantlings of ships. As the shell plating and certain parts of the deck and inner bottom plating have not been reduced in the design of the welded ship described in Chap. V, and as these items compose a large percentage of the weight of the ship, such reductions will make possible further savings in the weight and cost of welded ships.

6. Automatic arc welding makes possible very much greater speeds and a superior product by eliminating the human element. Great progress has been made in recent years in the development of various types of these welding machines, and as their field of application hardly has been scratched, further reductions in cost may be expected when their use becomes more general. Speeds of $27\frac{1}{2}$, 23 and 15 ft. per hour have been obtained for butt welds of $\frac{3}{8}$, $\frac{1}{2}$, and $\frac{5}{8}$ in. and a speed of 18 ft. per hour for fillet welds of $\frac{3}{8}$ in. These speeds can be increased further by the use of additional welding heads.¹

¹ Subsequent to the presentation of this paper, the Newport News Shipbuilding and Dry Dock Company has made $\frac{3}{16}$ -in. fillet welds in bulkhead construction at a welding speed of 103 ft. per hour, using one automatic

7. The principal savings by welding are made in the drawing room; therefore, design problems should not be left to shop men. This cannot be emphasized too strongly, for after the work has reached the shop it is too late.

8. The future of gas cutting is bound up intimately with the extensive utilization of arc welding in new construction and repair. This has been demonstrated throughout the designs submitted. In fact, some of these designs and methods of assembly would have been impracticable without this tool, and it could have been used more extensively in place of the present mechanical processes if a cheaper gas supply were available.¹

9. The gas, resistance, thermit, and forge-welding processes have certain advantages, and a working knowledge of all of these should be obtained by the engineer supervising the development of welding.

10. Welding development should be in the hands of competent engineers, and welding shop foremen should have a working knowledge of electricity and the generation and uses of the gases used in welding and cutting.

11. The advance of welding has been handicapped by a lack of satisfactory design values for welds and simple methods of showing welds on drawings. These have been covered fully in the Appendix and used in the designs and illustrations submitted, so that the reader may become familiar with them. As much of this information has been in use from one to two years in the design departments and shops of a large industrial organization and in the United States Navy, and has been tentatively approved by the Nomenclature, Definitions, and Symbols Committee of the American Welding Society, of which the author is chairman, it therefore can be used with confidence. Inspection must of necessity be very rigid for a number of years, but it will be aided by the use of the weld gages submitted and the shop requirements outlined.

12. It is recommended that the steel mills consider the rolling of larger tees and other sections required for economical welded construction. The need for such sections will be noted through-

welding head. The welding speed for butt and fillet welds given has also been increased.

¹ A liquefaction plant has recently been installed at the plant of the Newport News Shipbuilding and Dry Dock Company, making possible a reduction of at least 50 per cent in the cost of oxygen.

out the designs. T-sections, when welded to plating, form the ideal section for strength (that is, the I-section), but those now rolled are not large enough for general structural purposes. Structural shapes also will be wanted with square edges and with flanges that are not tapered. It has been found that flat bars that are used to replace angles are too flexible and are not only liable to damage in handling, but will not properly align themselves on the work; therefore, a section, which in effect is a flat bar with a very narrow flange on one edge merely for the purpose of giving stiffening to the bar, would be very desirable.

13. Arc welding is fully capable of standing on its own merits; therefore, if the cost of the product cannot be reduced or the product be improved by its use, then except in special cases the methods or process should be used that will give the desired results.

14. Although the advance of arc welding has been rapid during the last ten years, its future progress will be astounding. Indeed, the point already has been reached where it is necessary for the various bodies that govern construction—the building codes of cities, insurance companies, ship classification societies, and the many inspection services—to take cognizance of this new and powerful tool so as to keep up with its advancement and not to hinder progress.

15. Knowledge of arc welding is far from complete, but that which is available is ample for a sound and progressive development if guided by the engineering profession; and it is recommended that technical schools establish courses to supply the demand for capable engineers in this new, profitable, and interesting profession.

APPENDIX

ARC-WELDING DEFINITIONS, NOMENCLATURE, SYMBOLS, WELD GAGES, DESIGN DATA, FUNDAMENTAL SPECIFICATIONS, TENTATIVE STANDARDS FOR SHIPS' HULLS AND MACHIN- ERY, LIST OF ARC-WELDING APPLICATIONS TO SHIPS' HULLS AND MACHINERY, MISCELLA- NEOUS ILLUSTRATIONS

INTRODUCTION

The subject matter of this chapter was prepared originally and gages were designed for the use of the company with which the author is associated, and it embraced all of the welding processes. As it was necessary to restrict the subject of the paper to arc welding, this Appendix accordingly has been condensed to an extent that seemed appropriate. Some changes also have been made in the Definitions and in the Nomenclature divisions as the result of the author's work on the Nomenclature, Definitions and Symbols Committee of the American Welding Society, and improvements have been made in the Weld Gages division as suggested by H. H. Moss, of the Linde Air Products Company.

The symbols were developed to convey clearly to the shop the requirements of the designer and the weld gages to insure that these requirements are actually and consistently secured on the structure welded. These symbols and gages have been in practical use long enough to demonstrate their practical value in obtaining welded joints of the required efficiencies at the lowest cost. The gages are now in practical use not only in the plant of the Newport News Shipbuilding and Dry Dock Company but in several navy yards, the Linde Air Products Company, the Air Reduction Sales Company, the General Electric Company, and the Westinghouse Electric and Manufacturing Company, to which sample gages were furnished at the request of F. M. Farmer, president of the American Welding Society.

Design Data, Fundamental Specifications, and Tentative Standards for Ships' Hulls and Machinery are in such form that

they can be made use of readily by designers and shop personnel. The Fundamental Specifications division is a condensation or summary of the Fundamentals of Design and Shop Practice covered in Chap. II, and they are *applicable to arc-welded structures of all types*. The Standards, although tentative, will provide naval architects and engineers with type details of composite and completely welded joints that can be used in the hull structure and machinery parts of merchant and naval ships. The List of Welding Applications to Ships' Hulls and Machinery will show to what parts of a ship arc welding recently has been applied economically, and the Miscellaneous Illustrations will suggest numerous arc-welding applications impracticable to cover in the text.

The composite designs shown in the Tentative Standards will assist in the gradual application of arc welding to hull construction and the building up of a welding organization in shipyards, the ultimate object being the completely welded ship as described in Chap. V. When a tentative standard has been approved by the American Bureau of Shipping for a specific ship, a notation has been made to this effect. These standards, in general, also are applicable to all classes of arc-welded construction. Some of the design data, fundamental specifications, and nomenclature have been incorporated in the general specifications of the Bureau of Construction and Repair of the Navy.

Section 1

ARC-WELDING DEFINITIONS

NOTE.—As the various forms of joints and welds are clearly illustrated in "Nomenclature," their definitions have been omitted.

Weld.—The localized intimate union of metal parts in the plastic and/or molten states, with the application of mechanical pressure or blows; and in the molten or molten and vapor states, without the application of mechanical pressure or blows.

Welding Process.—The means or methods used to produce a weld.

Fusion Welding.—The process of joining metal parts in the molten or molten and vapor states without the application of mechanical pressure or blows. This process includes gas, arc, and thermit welding. It is sometimes incorrectly termed "autogenous" welding.

Fusion Weld.—A weld made by the fusion welding process.

Arc Welding.—A fusion-welding process wherein the welding heat is obtained from an electric arc between the base metal and an electrode or between two electrodes.

Metal-arc Welding.—An arc-welding process wherein the electrode used is a metal rod or wire, which when melted by the arc supplies the filler metal in the weld.

Shielded Metal-arc Welding.—A metal-arc-welding process wherein the arc is surrounded by and the molten metal bathed in hydrogen or other suitable gas.

Carbon-arc Welding.—An arc-welding process wherein a carbon electrode is used and filler metal, if required, is supplied by a welding rod as in gas welding.

NOTE.—Carbon is a generic term and includes all modifications of structure, electrical conductivity, etc., due to manufacturing processes. Graphite is a form of carbon.

Shielded Carbon-arc Welding.—A carbon-arc-welding process wherein the arc is surrounded by and the molten metal bathed in hydrogen or other suitable gas.

Atomic Hydrogen Welding.—A fusion-welding process wherein the heat of an electric arc, between two suitable electrodes, is used to disassociate molecular hydrogen into its atomic form, which on recombining in the molecular form gives up the energy required to disassociate it, producing a flame of very high temperature, at the same time bathing the molten weld metal in hydrogen. It may be considered as a combination of the gas- and arc-welding processes.

Manual Weld.—A weld made by an operator without the use of mechanical means.

Automatic Weld.—A weld made by automatic equipment.

Semi-automatic Weld.—A weld made partially by automatic equipment and partially by manual means.

Full Automatic Weld.—A weld made entirely by automatic equipment.

Base Metal.—The parent metal welded by cut.

Filler Metal.—Material specially prepared for addition to the weld in some forms of the fusion-welding process.

Deposited Metal.—Filler metal which has been melted by a fusion-welding process.

Weld Metal.—The material composing the weld.

Root.—The zone at the bottom of the cross-sectional space provided to contain a fusion weld.

Leg.—One of the fusion surfaces of a fillet weld.

Throat.—The minimum thickness of a fusion weld along a straight line passing through its root.

Electrode.—Filler metal, in wire or rod form, or a carbon, used as a terminal or terminals in an electric circuit for the purpose of producing an electric arc.

Flux.—Material used in welding to prevent the formation of oxides, nitrides, etc., in the weld and to eliminate those which have formed. In metal-arc welding it is also employed to retain the various elements of the electrode and to retard the rate of cooling of the weld metal.

Bare Electrode.—A metal electrode which is not fluxed or covered.

Welding Rod.—Filler metal, in wire or rod form, used in the gas- and those arc-welding processes wherein the electrode does not furnish the filler metal.

Fluxed Electrode.—A metal electrode provided with a flux.

Covered Electrode.—A metal electrode having an external wrapping or braiding of paper, asbestos, or other material. A flux may be included with the covering.

Coated Electrode.—A fluxed electrode having the flux applied externally by dipping, spraying, painting, or otherwise.

Flux Encased Electrode.—A fluxed electrode having the flux between a metal core and sheath.

Composite Electrode.—A fluxed electrode having one or more filler materials combined mechanically with the flux or covering.

Joint.—That portion of a structure wherein separate base metal parts are united. Welded joints may contain one or more independent welds.

Composite Joint.—A joint wherein both rivets and strength welds are used to unite the separate base metal parts.

Composite Structure.—A structure wherein both welded and riveted, welded and composite, or riveted and composite joints are employed.

Non-rigid Joint.—A joint wherein the parts can readily move and/or deform during welding so as to relieve thermal stresses (see Fig. 1).



FIG. 1.

Semi-rigid Joint.—An open butt joint used in the gas- and arc-welding processes wherein the joint edges are angularly placed prior to welding so as to relieve some of the thermal stresses by a gradual closing of the joint edges during welding. The weld is made in one continuous layer (see Fig. 2).

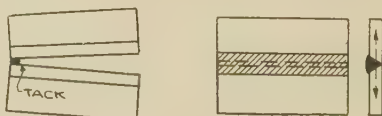


FIG. 2.

Rigid Joint.—A butt joint wherein the joint edges are tacked parallel prior to welding and at least one of the parts is unrestrained during welding (see Fig. 3).



FIG. 3.

Fixed Joint.—A butt joint wherein no movement and/or deformation of the parts is possible during welding except by preheating or by an elongation and/or compression of the parts (see Fig. 4).

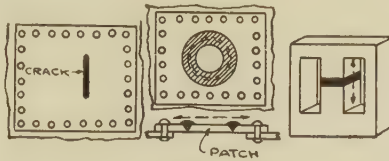


FIG. 4.

Continuous Weld.—A weld of unbroken continuity (see Fig. 5).

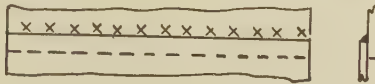


FIG. 5.

Intermittent Weld.—A weld of broken continuity, the increments of which are of equal length and equally spaced. The increments, in general, have a length of 1, 2, or 3 in. and a center-to-center distance of 4, 6, or 12 in. The crater is eliminated in measuring the length of the increment (see Fig. 6).

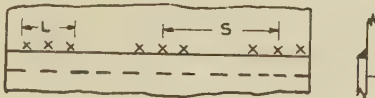


FIG. 6.

Tack Weld.—An intermittent weld used for assembly purposes only (see Fig. 7).

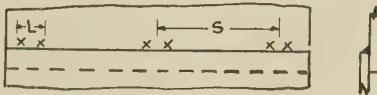


FIG. 7.

Strength Weld.—A weld having a predetermined design value and primarily intended to secure a mechanically strong joint. Its continuity may be either continuous or intermittent.

Calk Weld.—A continuous arc or gas weld primarily intended to secure tightness.

Composite Weld.—A weld complying with the requirements for both strength and calk welds.

Flush.—A term applied to the exposed surface of butt welds when the surface is even with the surfaces joined.

Concave.—A term applied to a butt weld when its throat thickness is less than the thickness of the thinner part joined, and to a fillet weld when its throat thickness is less than the thickness of the throat of a standard fillet weld.

Reinforcement.—Weld metal in excess of that required for standard fillet welds; and for butt welds, weld metal extending beyond a surface of the thinner part joined.

Flat Weld.—A butt or fillet weld made by the fusion-welding process with its linear direction horizontal or inclined at an angle of 45° or less to the horizontal, the weld being made downward in a general horizontal direction.

Vertical Weld.—A butt or fillet weld made by the fusion-welding process with its linear direction vertical or inclined at an angle less than 45° to the vertical, the weld being made horizontally in a general vertical direction.

Overhead Weld.—A butt or fillet weld made by the fusion-welding process with its linear direction horizontal or inclined at an angle less than 45° to the horizontal, the weld being made upward in a general horizontal direction.

Horizontal Weld.—A bead or a butt weld made by the fusion-welding process with its linear direction horizontal or inclined at an angle less than 45° to the horizontal, the weld being made horizontally in a general horizontal direction.

Residual Stress.—The stress or stresses remaining in a welded joint, welded structure, or part cut on completion of welding or cutting.

Weld Size.—The cross-sectional dimensions of a weld. The size of a fillet weld is the length in inches of its legs. The size of a butt weld is the net or unreinforced throat dimension in inches.

Weld Length.—The unbroken length of the full cross-section of a weld, exclusive of the length of any craters.

Electrode Holder.—A tool used in the arc-welding process for mechanically holding and conducting the electric current to the electrode.

Face Shield.—A protective device used in arc welding for shielding the operator's face, neck, and eyes, provided with a window having a protective lens and designed to be held in front of the face by the hand.

Helmet.—A protective device used in arc welding for shielding the operator's face, neck, and eyes, provided with a window having a protective lens and designed to be held in place by the operator's head.

Motor-generator Set.—An electrical device consisting of one or more motors mechanically coupled to one or more generators.

Single-operator Motor-generator Set.—A motor-generator set designed to supply current to only one welding arc.

Multiple-operator Motor-generator Set.—A motor generator set designed to supply current to two or more welding arcs at the same time.

Resistor.—A device possessing the property of electrical resistance and used in arc-welding circuits for the purpose of regulating the arc amperes.

Reactor.—A device possessing the property of reactance and used in direct-current arc-welding circuits to stabilize the arc.

Constant-voltage Welding Source.—A source which automatically maintains its voltage within 5 per cent of the rated full-load setting over the range from full load to no load and has a time of recovery of not more than 0.3 sec.

Variable-voltage Welding Source.—A source the voltage of which automatically reduces as the current increases, but not in such a way as to qualify the source as a constant-energy source.

Constant-current Welding Source.—A source which, when adjusted to give rated current output with normal arc voltage, will automatically maintain this current output within 5 per cent of the rated value with a variation in arc voltage of 10 per cent above or below the normal arc voltage and has a time of recovery of not more than 0.3 sec.

Constant-power Welding Source.—A source which when adjusted to give rated power output with normal arc voltage will automatically maintain this power output within 5 per cent of the rated value with a variation in arc voltage of 10 per cent above or below the normal arc voltage and has a time of recovery of not more than 0.3 sec.

NOMENCLATURE



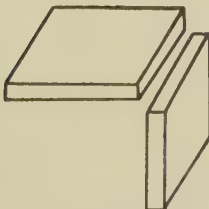
4. Edge joint



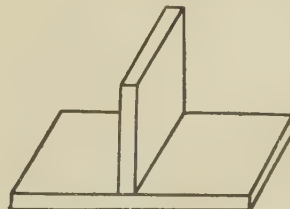
B. Butt joint



C. Lap joint



D. Corner joint



E. Tee joint

FIG. 8.—Fundamental forms of joints.

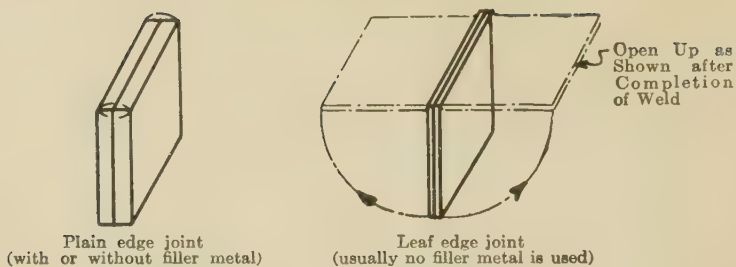


FIG. 9.—Various forms of edge joints.

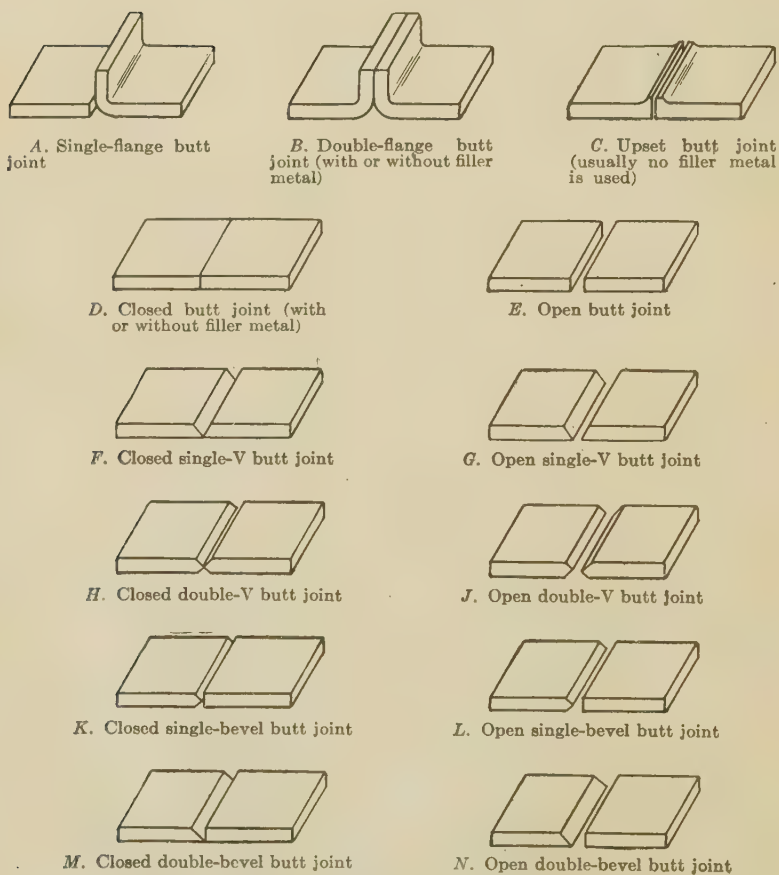


FIG. 10.—Various forms of butt joints.



A. Strapped closed butt joint



B. Strapped open butt joint



C. Strapped closed single-V butt joint



D. Strapped open single-V butt joint

FIG. 11.—Various forms of strapped butt joints.



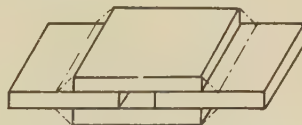
A. Single lap joint



B. Double lap joint



C. Single parallel lap joint



D. Double parallel lap joint



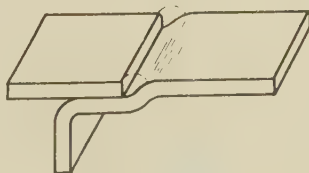
E. Closed-joggle lap joint



F. Open-joggle lap joint



G. Single-flange lap joint



H. Single-flange-joggle lap joint

FIG. 12.—Various forms of lap joints.

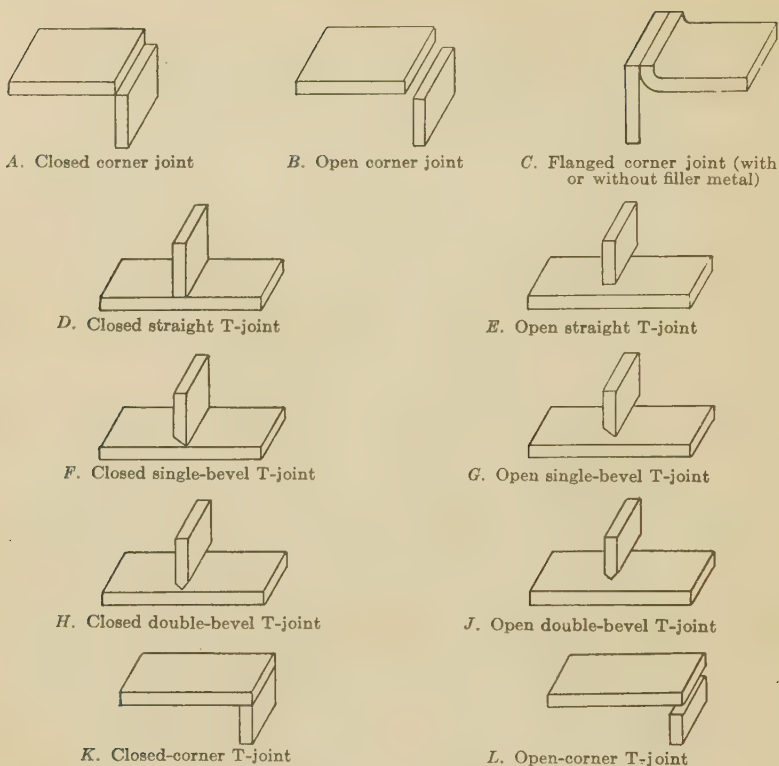


FIG. 13.—Various forms of corner and T-joints.

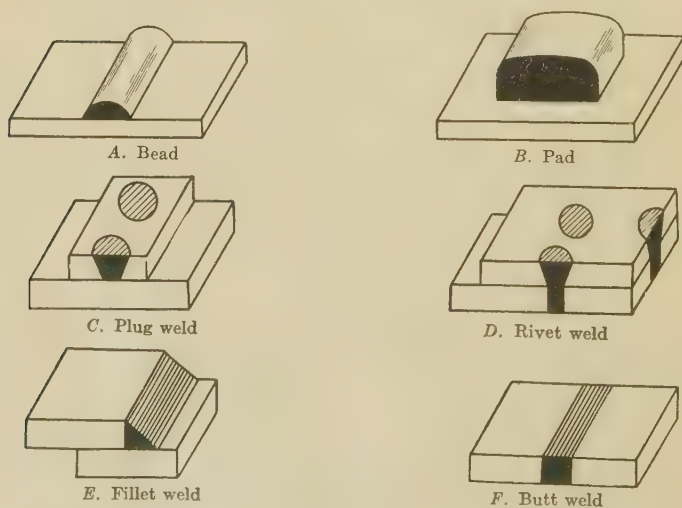
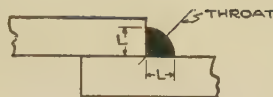


FIG. 14.—Fundamental forms of fusion welds.



A. Standard fillet weld. (Size of weld specified in terms of L . L = not less than $1.42 \times$ throat required)



B. Standard reinforced fillet weld. (Size of weld specified in terms of L . L = not less than throat required)



C and D. Light fillet weld. (Size of weld specified thus: "Light." Throat = $0.35 T$, L = not less than $0.50 T$)



E and F. Full fillet weld. (Size of weld specified thus: "Full." Throat = $0.070 T$, L = not less than T)



G and H. Reinforced light fillet weld. (Size of weld specified thus: LR . Throat = $0.50 T$, L = not less than $0.50 T$)



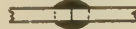
J and K. Reinforced full fillet weld. (Size of weld specified thus: FR . Throat = T , L = not less than T)

FIG. 15.—Various forms of fillet welds.

NOTE: T = thickness of thinner section joined.



A. Closed butt weld (reinforced one side)



B. Closed butt weld (reinforced both sides)



C. Open butt weld (reinforced one side)



D. Open butt weld (reinforced both sides)



E. Open single-V butt weld (reinforced one side)



F. Open single-V butt weld (reinforced both sides)



G. Closed single-V butt weld (reinforced one side)



H. Open double-V butt weld (reinforced both sides)

FIG. 16.—Various forms of butt welds.

Section 2

SYMBOLS

METHODS OF SPECIFYING WELDING PROCESSES AND WELDS ON DRAWINGS

TABLE 1.—NAMES OF THE PRINCIPAL WELDING PROCESSES IN FULL

Forge welding		Smith welding Hammer welding Roll welding	
Pressure welding	Resistance welding	Butt welding	Flash welding Upset welding
		Spot welding } (see note No. 2) Seam welding } Percussive welding	
	Thermit pipe welding		
Fusion welding	Gas welding (see note No. 4)	Atomic welding	
	Arc welding	Carbon-arc welding	Shielded carbon-arc welding
		Metal-arc welding	Shielded metal-arc welding
	Thermit welding (see note No. 3)		

Notes:

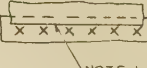
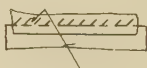
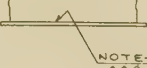
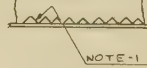
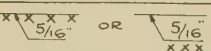
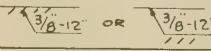
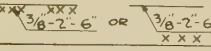
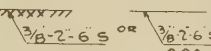
1. If the welding process to be used is not obvious, give name of process in full.
2. The use of the terms "spot" and "seam" shall be restricted to the resistance-welding process.
3. When the thermit-welding process is referred to, regular thermit welding is understood, unless thermit pipe welding, thermit roll-neck welding, etc., is specified.
4. When the gas welding process is referred to, "oxy-acetylene" welding is understood unless "oxy-hydrogen" etc., is specified.

TABLE 2.—WELDING ABBREVIATIONS

<i>W</i>	Weld
<i>J</i>	Joint
<i>WJ</i>	Welded joint
<i>R</i>	Reinforced weld
<i>P</i>	Plug weld
<i>C</i>	Continuous weld
<i>INT</i>	Intermittent weld
<i>B</i>	Bead
<i>S</i>	Staggered
<i>1L</i>	One layer (see Note 2)
<i>2L</i>	Two layers (see Note 2)
<i>H</i>	Each layer hammered or peened, (see Note 3)
<i>1H</i>	First layer hammered or peened, (see Note 3)
<i>2H</i>	First and second layers hammered or peened, (see Note 3)
<i>F</i>	Finished weld
<i>FG</i>	Weld finished by grinding
<i>FH</i>	Weld finished by hammering or peening
<i>FM</i>	Weld finished by machining

Notes:

1. Fillet and butt welds used in the gas- and arc-welding processes shall be considered as continuous unless otherwise specified.
2. All welds made by the gas- and arc-welding processes are understood to be made in one layer, unless multiple layers are specified.
3. All welds made by the gas- and arc-welding processes are understood to be made without mechanical or thermal treatment unless otherwise specified.

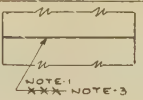
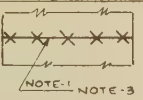
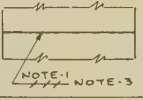
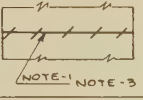
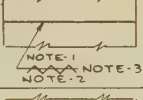
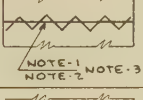
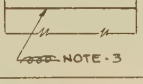
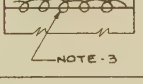
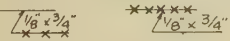
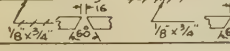
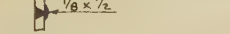
FIG. No	SYMBOL	SYMBOLS AS USED IN PLAN OR ELEVATION		METHOD USED FOR SECTIONS
		METHOD No. 1 (PREFERABLE FOR ALL SCALES)	METHOD No. 2 (MAY BE USED FOR SCALES $\frac{1}{2}$ AND ABOVE)	
I	XXXX FILLET WELD ON NEAR SIDE OF JOINT	 NOTE-1 XXX	 NOTE-1	 NOTE-1
II	//// FILLET WELD ON FAR SIDE OF JOINT	 NOTE-1 ///	 NOTE-1	 NOTE-1
III	~~~~~ FILLET WELD ON BOTH SIDES OF JOINT	 NOTE-1 ~~~	 NOTE-1	 NOTE-1
NOTE-1 GIVE SIZE AND CONTINUITY OF WELD HERE AS SHOWN BY THE FOLLOWING EXAMPLES.				
EXAMPLE		DESCRIPTION		
		$\frac{5}{16}$ " STANDARD CONTINUOUS FILLET WELD ON NEAR SIDE OF JOINT.		
		$\frac{3}{8}$ " STANDARD CONTINUOUS FILLET WELD 12" LONG ON FAR SIDE OF JOINT.		
		$\frac{1}{2}$ " STANDARD REINFORCED CONTINUOUS FILLET WELD ON FAR SIDE OF JOINT.		
		$\frac{3}{8}$ " STANDARD INTERMITTENT FILLET WELD ON NEAR SIDE OF JOINT HAVING INCREMENTS 2" LONG SPACED 6" C-C.		
		$\frac{3}{8}$ " STANDARD INTERMITTENT FILLET WELD ON BOTH SIDES OF JOINT HAVING INCREMENTS 2" LONG SPACED 6" C-C ON EACH SIDE AND WELDS STAGGERED WITH RESPECT TO EACH OTHER		
		$\frac{3}{8}$ " STANDARD CONTINUOUS FILLET WELD.		

NOTE-2

USE A SOLID DOT IN CONJUNCTION WITH OTHER INFORMATION TO INDICATE A WELD WHICH IS SPECIFICALLY REQUIRED TO BE MADE IN THE FIELD. THUS



17.—Conventional methods of showing on drawings the necessary information required to make fillet welds.

FIG. No.	SYMBOL	SYMBOLS AS USED IN PLAN OR ELEVATION		METHOD USED FOR SECTIONS
		METHOD NO. 1 (PREFERABLE FOR ALL SCALES)	METHOD NO. 2 (MAY BE USED FOR SCALES 1/2 AND ABOVE)	
I	XXXXX REINFORCEMENT ON NEAR SIDE OF JOINT.			
II	+++++ REINFORCEMENT ON FAR SIDE OF JOINT.			
III	~~~~~ REINFORCEMENT ON BOTH SIDES OF JOINT. NOTE-THIS IS THE STANDARD METHOD OF REINFORCEMENT			
IV	OOOOO WELD FLUSH ON BOTH SIDES OF JOINT NOTE- ONLY TO BE USED BY SPECIAL PERMISSION.			
NOTE-				
1. GIVE SIZE OF REINFORCEMENT HERE IN TERMS OF HEIGHT AND WIDTH.				
2. GIVE SIZE OF REINFORCEMENT ON FAR SIDE HERE, IF DIFFERENT FROM REINFORCEMENT ON NEAR SIDE.				
3. MAKE FREE HAND SKETCH OF JOINT HERE, IF SHAPE OF JOINT EDGES, SPACING BETWEEN EDGES AND SIDE FROM WHICH BEVELED IS NOT OBVIOUS. THE UPPER SIDE OF THE SKETCH WILL BE UNDERSTOOD TO BE THE NEAR SIDE.				
EXAMPLE		DESCRIPTION		
		REINFORCEMENT 1/8" HIGH AND 3/4" WIDE ON NEAR SIDE OF JOINT.		
		BUTT JOINT HAVING A 1/8 x 3/4" REINFORCEMENT ON NEAR SIDE AND A 1/8 x 1/2" REINFORCEMENT ON FAR SIDE.		
		60° SINGLE V BUTT JOINT, BEVELED FROM FAR SIDE A 3/16" OPENING BETWEEN EDGES AND A 1/8 x 3/4" REINFORCEMENT ON FAR SIDE.		
		SINGLE V BUTT JOINT WITH A 1/8 x 1/2" REINFORCEMENT ON BOTTOM OF V		

NOTE

4. USE A SOLID DOT IN CONJUNCTION WITH OTHER INFORMATION TO INDICATE A WELD WHICH IS SPECIFICALLY REQUIRED TO BE MADE IN THE FIELD. THUS:-



FIG. 12.—Conventional methods of showing on drawings the necessary information required to make butt welds.

Section 3

WELD GAGES

FOR MEASURING FILLET WELDS AND THE REINFORCEMENT OF BUTT WELDS MADE BY THE ARC-WELDING PROCESS

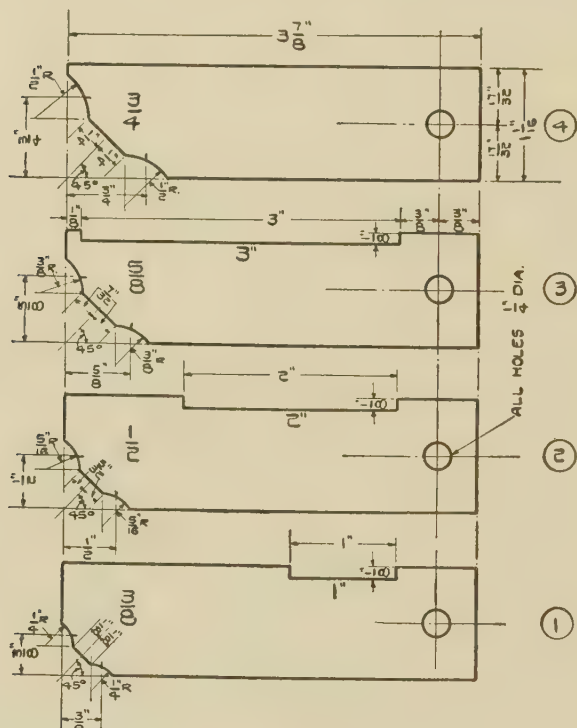


FIG. 19.—Weld gage for standard and intermittent fillet welds.

NOTE: All blades are 0.0625 in. thick and made of monel metal, brass, or stainless steel.

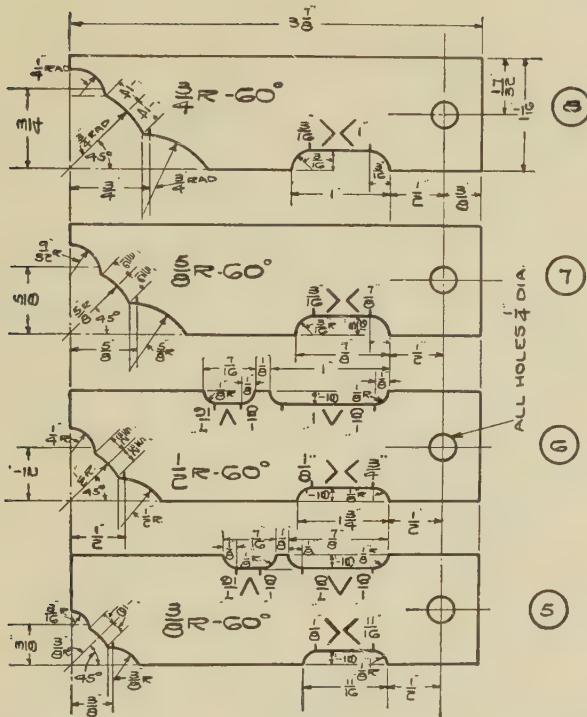
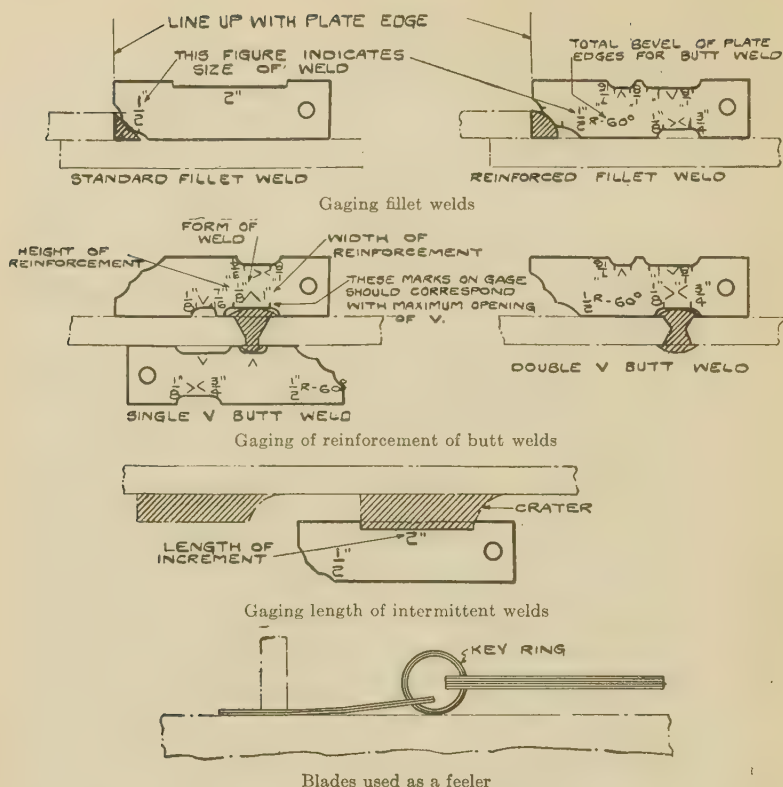


FIG. 20.—Weld gage for reinforced fillet welds and reinforcement of butt welds.

NOTE: All blades are 0.0625 in. thick and made of monel metal, brass, or stainless steel.



Blades used as a feeler
FIG. 21.—Method of using weld gage.

NOTE: As each blade is 0.0625 in. thick, amount of opening is determined by number of blades required.

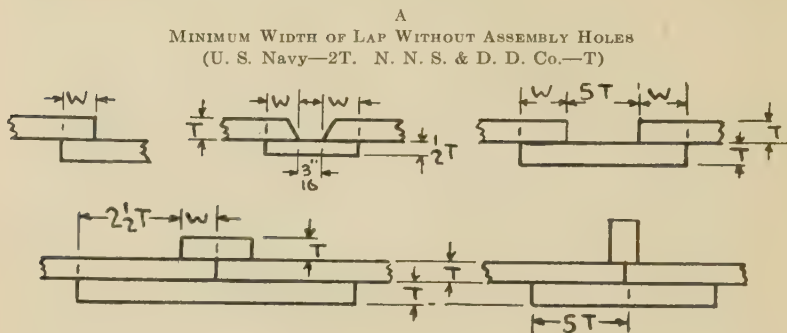
Section 4

DESIGN DATA

TABLE 3.—MINIMUM WIDTH OF LAP FOR COMPOSITE JOINTS
(Width of standard single-riveted lap)

U. S. Navy				American Bureau				Lloyds			
Weight of plate (pounds)	Thickness of plate (inches)	Diam-eter hole (inches)	Width lap (inches)	Weight of plate (pounds)	Thickness of plate (inches)	Diam-eter hole (inches)	Width lap (inches)	Weight of plate (pounds)	Thickness of plate (inches)	Diam-eter hole (inches)	Width lap (inches)
Under 3.....		$\frac{5}{16}$	$1\frac{3}{16}$								
From 3 to 7 incl.....		$\frac{3}{16}$	$1\frac{1}{4}$								
Over 7 to $8\frac{1}{2}$ incl....		$\frac{9}{16}$	$1\frac{5}{8}$								
Over $8\frac{1}{2}$ to $12\frac{1}{2}$ incl..		$1\frac{1}{4}$	$2\frac{1}{16}$	7.34 to 13.87	0.18 to 0.34	$1\frac{1}{16}$	$2\frac{1}{4}$	14.69	Not over 0.36	$1\frac{1}{16}$	$2\frac{1}{4}$
Over $12\frac{1}{2}$ to 19 incl....	0.31 to 0.46	$1\frac{3}{16}$	$2\frac{1}{16}$	14.59 to 15.5	0.36 to 0.48	$1\frac{3}{16}$	$2\frac{1}{2}$	14.69 to 20.4	Over 0.36 to 0.50 incl.	$1\frac{3}{16}$	$2\frac{1}{2}$
Over 19 to 29 incl....	0.47 to 0.72	$1\frac{5}{16}$	$2\frac{3}{8}$	20.4 to 27.7	0.50 to 0.68	$1\frac{5}{16}$	3	20.4 to 29.4	Over 0.50 to 0.72 incl.	$1\frac{5}{16}$	3
Over 29 to 39 incl....	0.73 to 0.96	$1\frac{7}{16}$	$3\frac{1}{4}$	28.56 to 35.9	0.70 to 0.88	$1\frac{7}{16}$	$3\frac{1}{2}$	29.4 to 38.3	Over 0.72 to 0.94 incl.	$1\frac{7}{16}$	$3\frac{1}{4}$
Over 39 to 49 incl....	0.97 to 1.18	$1\frac{9}{16}$	$3\frac{1}{4}$	36.7 to 45.6	0.90 to 1.12	$1\frac{9}{16}$	4	38.3 to 45.7	Over 0.94 to 1.14 incl.	$1\frac{9}{16}$	$3\frac{1}{2}$
Over 49.....	1.19	$1\frac{11}{16}$	$4\frac{1}{16}$	45.7 to 51.4	1.14 to 1.26	$1\frac{11}{16}$	$4\frac{1}{4}$	4.57 to 51.4	Over 1.14 to 1.26 incl.	$1\frac{11}{16}$	4

TABLE 4.—MINIMUM WIDTHS OF LAPS AND THICKNESS OF STRAPS FOR WELDED JOINTS

NOTE: T = thickness of thinner section joined.

B

MINIMUM WIDTH OF LAP WITH OR WITHOUT ASSEMBLY HOLES

Classification society	Plate		Width of lap "W" (inches)
	Weight (pounds per square foot)	Thickness (inches)	
American Bureau of Shipping.....	All	All	$2T + 1$
Lloyds Register of Shipping	16.32	0.40 and under	$2\frac{1}{4}$
	24.48	0.60	$2\frac{1}{2}$
	32.64	0.80	$2\frac{3}{4}$
	40.80	1.00	3

C

MINIMUM WIDTH OF LAP WITH ASSEMBLY HOLES
(Newport News Shipbuilding and Dry Dock Co.)

Plate		Assembly holes		Width of lap "W"	
Weight (pounds per square foot)	Thickness (inches)	Diameter (inches)	Punched or drilled	Flat surface (inches)	Curved surfaces difficult to assemble (inches)
9.8	0.24	$\frac{9}{16}$	Punched	$1\frac{1}{4}$	$1\frac{3}{8}$
10.6 -13.87	0.26-0.34	$1\frac{1}{16}$	Punched	$1\frac{3}{8}$	$1\frac{1}{2}$
14.68-19.58	0.36-0.48	$1\frac{3}{16}$	Punched	$1\frac{1}{2}$	$1\frac{5}{8}$
20.40-28.56	0.50-0.70	$1\frac{5}{16}$	Punched	$1\frac{3}{4}$	$1\frac{7}{8}$
29.37-34.27	0.72-0.84	$1\frac{1}{2}$	Punched	2	$2\frac{1}{8}$
35.08-37.53	0.86-0.92	$1\frac{1}{2}$	Drilled	2	$2\frac{1}{8}$
		$1\frac{3}{16}$	Punched	$2\frac{1}{8}$	$2\frac{1}{4}$
38.35-39.98	0.94-0.98	$1\frac{1}{2}$	Drilled	2	$2\frac{1}{8}$
		$1\frac{5}{16}$	Punched	$2\frac{1}{4}$	$2\frac{5}{8}$
40.80-47.33	1.00-1.16	$1\frac{3}{16}$	Drilled	$2\frac{1}{4}$	$2\frac{5}{8}$
		$1\frac{1}{2}$	Punched	$2\frac{3}{8}$	$2\frac{1}{2}$

References:

American Bureau of Shipping Rules, 1927 Edition, Sec. 26.

Lloyds Register of Shipping, 1925-26 Edition—page 84.

Navy Dept. Specifications, Appendix 21, 1927, pages 6 and 13.

TABLE 5.—SAFE LOAD PER LINEAR INCH OF FILLET WELDS

Thickness of base metal (inches)		Depth of throat (inches)			Safe load in pounds per linear inch of a fillet weld					
					Throat $\times 9,000^1$ (for general use)			Throat $\times 10,000^2$		
Fraction	Decimal	Light	Full	Reinforced	Light	Full	Reinforced	Light	Full	Reinforced
$\frac{1}{8}$	.1250	.044	.088	.1250	396	792	1,125	440	880	1,250
$\frac{3}{16}$	.1875	.067	.133	.1875	603	1,197	1,688	665	1,330	1,875
$\frac{1}{4}$	.2500	.088	.177	.2500	792	1,593	2,250	885	1,770	2,500
$\frac{5}{16}$	.3125	.111	.221	.3125	999	1,989	2,813	1,105	2,210	3,125
$\frac{3}{8}$	.3750	.133	.265	.3750	1,196	2,385	3,375	1,325	2,650	3,750
$\frac{7}{16}$	.4375	.155	.310	.4375	1,395	2,790	3,938	1,550	3,100	4,375
$\frac{1}{2}$	.5000	.177	.354	.5000	1,593	3,186	4,500	1,770	3,540	5,000
$\frac{9}{16}$	.5625	.199	.398	.5625	1,791	3,582	5,063	1,990	3,980	5,625
$\frac{5}{8}$	.6250	.221	.442	.6250	1,989	3,978	5,625	2,210	4,420	6,250
$\frac{11}{16}$	.6875	.243	.486	.6875	2,187	4,374	6,188	2,430	4,860	6,875
$\frac{3}{4}$	.7500	.265	.530	.7500	2,385	4,770	6,750	2,650	5,300	7,500
$\frac{13}{16}$	.8125	.285	.570	.8125	2,565	5,130	7,313	2,850	5,700	8,125
$\frac{7}{8}$	.8750	.310	.619	.8750	2,790	5,571	7,875	3,095	6,190	8,750
$\frac{15}{16}$	.9375	.334	.667	.9375	3,006	6,003	8,438	3,335	6,670	9,375
1	1.0000	.354	.707	1.0000	3,186	6,363	9,000	3,535	7,070	10,000
$1\frac{1}{8}$	1.1250	.398	.796	1.1250	3,582	7,164	10,125	3,980	7,960	11,250
$1\frac{1}{4}$	1.2500	.442	.884	1.2500	3,978	7,956	11,250	4,420	8,840	12,500
$1\frac{1}{2}$	1.5000	.530	1.061	1.5000	4,770	9,549	13,500	5,305	10,610	15,000
$1\frac{3}{4}$	1.7500	.619	1.237	1.7500	5,571	11,133	15,750	6,135	12,370	17,500
2	2.0000	.707	1.414	2.0000	6,363	12,726	18,000	7,070	14,140	20,000

¹ Weld metal 36,000 lb. per square inch using a factor of safety of 4.² Weld metal 40,000 lb. per square inch using a factor of safety of 4.

TABLE 6.—FILLET WELD REQUIRED TO DEVELOP FULL STRENGTH OF ONE RIVET IN SHEAR

Thickness of base metal (inches)		Diameter of rivet (inches)	Ultimate strength of fillet weld per linear inch		Linear inches of continuous fillet weld required to develop full strength of one rivet			
Fraction	Decimal		Full	Reinforced	Single shear		Double shear	
					Full	Reinforced	Full	Reinforced
1		2	3	4	5	6	7	8
$\frac{1}{8}$	0.1250	$\frac{3}{8}$	3,168	4,500	*2.07	*1.46	*2.07	*1.46
$\frac{3}{16}$	0.1875	$\frac{1}{2}$	4,788	6,752	*2.64	*1.87	*2.64	*1.87
$\frac{1}{4}$	0.2500	$\frac{5}{8}$	6,372	9,000	2.91	2.06	*3.24	*2.29
$\frac{5}{16}$	0.3125	$\frac{3}{4}$	7,956	11,252	2.33	1.65	*3.24	*2.29
$\frac{3}{8}$	0.3750	$\frac{7}{8}$	9,520	13,500	2.72	1.92	*3.84	*2.71
$\frac{7}{16}$	0.4375	$\frac{1}{2}$	11,160	15,752	2.32	1.65	*3.83	*2.71
$\frac{1}{2}$	0.5000	$\frac{3}{4}$	12,744	18,000	2.71	1.92	*4.42	*3.13
$\frac{9}{16}$	0.5625	$\frac{7}{8}$	14,328	20,252	2.41	1.71	4.33	3.01
$\frac{5}{8}$	0.6250	$\frac{1}{2}$	15,912	22,500	2.17	1.54	3.90	2.76
$\frac{11}{16}$	0.6875	$\frac{3}{4}$	17,496	24,752	1.97	1.39	3.55	2.51
$\frac{3}{4}$	0.7500	1	19,080	27,000	2.32	1.65	4.18	2.96
$\frac{13}{16}$	0.8125	1	20,520	29,252	2.16	1.52	3.89	2.73
$\frac{7}{8}$	0.8750	1	21,884	31,500	2.03	1.41	3.65	2.53
$\frac{15}{16}$	0.9375	$1\frac{1}{8}$	24,012	33,752	2.31	1.65	4.15	2.95
1	1.0000	$1\frac{1}{2}$	25,452	36,000	2.18	1.54	3.92	2.77

* Bearing value determines strength.

NOTES

1. Tensile strength of base metal per square inch..... 60,000 lb.
Tensile strength of weld metal per square inch..... 36,000 lb.
Shearing value of rivets per square inch single shear..... 50,000 lb.
Shearing value of rivets per square inch double shear ($1.8 \times 50,000$)... 90,000 lb.
Bearing value of plate per square inch..... 120,000 lb.
2. All values given are ultimate and any factor of safety used would not affect the comparison.
3. Linear inches of weld metal required to develop full shearing strength of one rivet is based on the value which determines the strength of the connection. (Diameter of rivet hole used.)
4. For linear inches of light continuous welding required to develop a relative strength, multiply columns 5 or 7 by 2.
5. One linear inch of base metal requires 2.357 linear inches of a full continuous fillet weld, or 1.666 linear inches of a reinforced continuous fillet weld to develop full strength of the base metal.

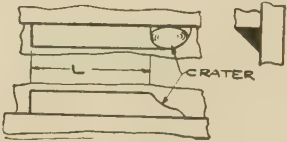
TABLE 7.—PERCENTAGE OF EFFICIENCY OF FILLET WELDS

Reference No.	Ultimate strength, (pounds per square inch)		Continuity of weld												
			Continuous			Intermittent									
	Weld metal	Base metal	Light	Full	Rein- forced	6 in. c. to c.					12 in. c. to c.				
						1 in.	1½ in.	2 in.	2½ in.	3 in.	2 in.	3 in.	4 in.	5 in.	6 in.
1		50,000	25.5	50.9	72.0	8.5	12.7	17.0	21.2	25.5	8.5	12.7	17.0	21.2	25.5
2		55,000	23.1	46.2	65.5	7.7	11.6	15.4	19.3	23.1	7.7	11.6	15.4	19.3	23.1
3		60,000	21.2	42.4	60.0	7.1	10.6	14.1	17.7	21.2	7.1	10.6	14.1	17.7	21.2
4	36,000	63,000	20.2	40.4	57.1	6.7	10.1	13.5	16.8	20.2	6.7	10.1	13.5	16.8	20.2
5		63,000	19.5	39.1	55.4	6.0	9.8	13.0	16.3	19.5	6.0	9.8	13.0	16.3	19.5
6		65,000	19.5	39.1	55.4	6.0	9.8	13.0	16.3	19.5	6.0	9.8	13.0	16.3	19.5
7		70,000	18.2	36.4	51.4	6.1	9.1	12.1	15.2	18.2	6.1	9.1	12.1	15.2	18.2
8															
9															
10															
11															
12		60,000	23.6	47.1	66.7	7.9	11.8	15.7	19.6	23.6	7.9	11.8	15.7	19.6	23.6
13	40,000	63,000	22.5	44.9	63.5	7.5	11.2	15.0	18.7	22.5	7.5	11.2	15.0	18.7	22.5
14															
15															
16															

Reference No.	Used by	Class of work
4	N. N. S. & D. D. Co.	Steel building structures Ships under A. B. S. classification Sharon Bldg. of Westinghouse Elec. & Mfg. Co.
12	American Bridge Co.	

TABLE 8.—LINEAR INCHES OF A STANDARD FILLET WELD REQUIRED TO DEVELOP FULL STRENGTH OF ONE BOLT

Diam- eter of bolt, inches	Ultimate strength of bolt, pounds	Linear inches of continuous fillet weld "L" required to develop full strength of one bolt for various sized welds															
		1⁄8"	3⁄16"	1⁄4"	5⁄16"	3⁄8"	7⁄16"	1⁄2"	9⁄16"	5⁄8"	1 1⁄16"	3⁄4"	13⁄16"	7⁄8"	1 1⁄8"	1"	
3⁄8	2,800	7⁄8	5⁄8	3⁄4													
1⁄2	5,200		1 1⁄16	7⁄8	1 1⁄16	5⁄8											
5⁄8	8,300			1 3⁄8	1 1⁄16	7⁄8	3⁄4	1 1⁄16									
3⁄4	12,400				1 9⁄16	1 3⁄8	1 1⁄8	1	1 1⁄16	7⁄8	1 3⁄16						
7⁄8	17,200				2 3⁄16	1 7⁄8	1 9⁄16	1 3⁄8	1 1⁄4	1 9⁄16	1 1⁄8	1	7⁄8				
1	22,600				2 7⁄8	2 9⁄16	2	1 3⁄4	1 9⁄16	1 7⁄16	1 9⁄16	1 3⁄16	1 1⁄8	1	1 9⁄16	7⁄8	



NOTES

1. Tensile strength of bolt material per square inch..... 41,000 lb.
Tensile strength of weld metal per square inch..... 36,000 lb.
2. All values given are ultimate and any factor of safety used would not affect the comparison.
3. Linear inches of weld metal required to develop full shearing strength of one bolt is based on section of bolt at root of thread.
4. Length of weld given excludes crater.

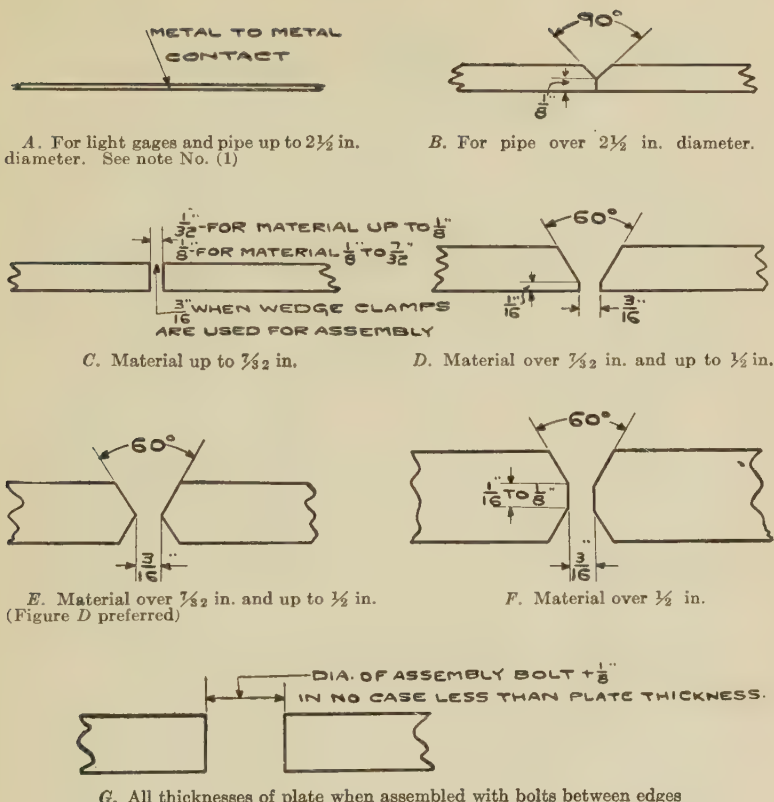


FIG. 22.—Angle of bevel and opening for metal-arc-welded joints.

NOTE: (1) Used for material up to approximately $\frac{1}{2}$ in. thick when welded by the automatic carbon arc and sometimes by the automatic metal arc.

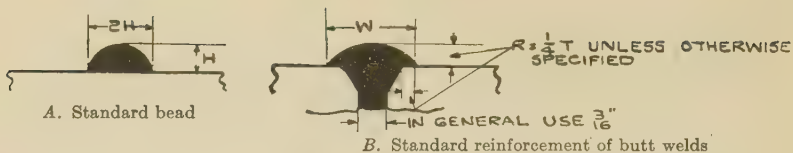


FIG. 23.—Standard bead and reinforcement of butt welds.

NOTE: Butt welds shall always be reinforced on both sides unless otherwise specified. Size or reinforcement specified in terms of $R \times W$.

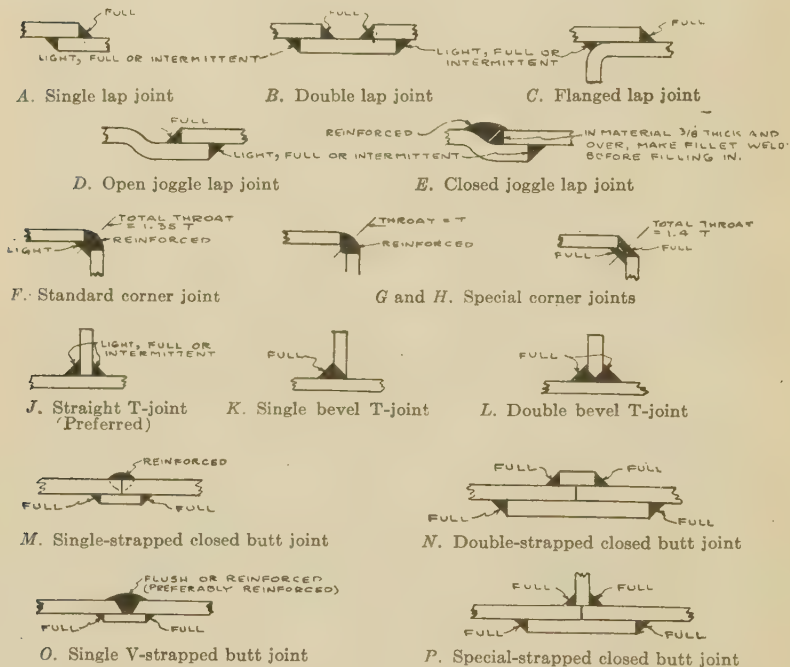


FIG. 24.—Welds in various forms of arc-welded joints.

Section 5

FUNDAMENTAL ARC-WELDING SPECIFICATIONS FOR ALL CLASSES OF STRUCTURAL WORK

NOTE.—Direct-current metal-arc welding of low- and medium-carbon steel plates, shapes, pipes, and low-carbon forgings and castings shall be understood unless otherwise stated.

1. *Galvanizing*.¹—If galvanizing is necessary, do so after welding. When necessary to weld galvanized material, remove coating in way of weld by grinding, sand-blasting, or sulphuric acid. Traces of sulphuric acid shall be removed by lime water.

2. *Automatic Welding*.—Automatic metal and carbon-arc welding shall be considered for all repeat production jobs of low- and medium-carbon-steel material.

3. *Reduction in Labor*.

(a) Order material neat. This is frequently possible when lap joints are used.

(b) Design to reduce handling to a minimum.

(c) Design to reduce operations to a minimum.

(d) Design to eliminate assembly bolts or reduce same to a minimum.

4. *Reduction in Weight*.

(a) Use flat bars for stiffening members instead of angles, channels, etc.

(b) Omit connecting angles and other flanged connections.

(c) Use narrow laps.

(d) As welded joints can be made of higher efficiency than riveted joints, it is frequently possible to use material of less weight than would be possible if riveted.

(e) In the substitution of welded steel for cast iron, the thickness of the metal can in general be made one-fourth that required for a casting. If stiffness is of primary importance, the welded steel should be approximately one-third that required for cast iron.

5. *Shop versus Field Welding*.—Design to have work done in the shop in preference to the field. In the design of building structures clearly show field welds.

6. *Position of Welding*.—Design so that the position of welding will be in the following order of preference: Flat, vertical, horizontal, overhead.

7. *Composite Design*.

(a) In general avoid using welding and riveting in the same joint. In joints requiring a large number of assembly bolts, this design would be satisfactory and economical; rivets, however, should preferably be driven on completion of welding.

(b) In structures in which both riveted and welded joints are employed, consideration should be given to the possibility of slippage of the riveted joints, thus causing the welded joints to take a greater proportion of the load than originally designed to stand, with the possibility of failure.

8. *Welding Light Material to Heavy Material*.—Consider carefully the welding of light to heavy material. The welding of $\frac{1}{8}$ in. metal to heavier

¹ See footnote on Galvanizing in Chap. II.

metal is entirely satisfactory if spot welded or if a fillet weld is made on the edge of the thinner part. Should a fillet weld be desired on the edge of the heavier part, the lighter metal shall preferably be not less than $\frac{1}{4}$ in. in thickness.

9. *Carbon Content of Material to Be Welded.*—The carbon content shall not in general exceed 0.25 per cent. A carbon content not exceeding 0.15 per cent is preferable.

10. *Types and Forms of Joints to Be Used.*—In general use the non-rigid type of joint (lap, tee, corner, etc.) in preference to the semi-rigid, rigid, and fixed types of butt joints. A butt joint should be used for the longitudinal joints of pressure vessels subjected to breathing stresses.

11. *Reinforcement of Welds.*

Butt Welds:

(a) Do not use butt welds with both surfaces flush except by special permission.

(b) Butt welds in strength joints shall in general be reinforced on both sides.

(c) Butt welds in pipe lines which cannot be reinforced on the inside of pipe may be reinforced on outside surface only.

Fillet Welds:

(d) Only use reinforced fillet welds when maximum strength is essential; when only possible to weld on one edge of a lap or tee joint and a standard fillet does not give required strength; and when reinforcement is necessary to take care of corrosion.

12. *Opening between Joint Edges.*

(a) The opening between joint edges of double lap and double parallel lap joints shall not be less than five times the thickness of the thinner material welded.

(b) The opening between joint edges of corner, single bevel tee, double bevel tee, single V butt and double V butt joints shall be $\frac{1}{8}$ to $\frac{3}{16}$ in.

(c) The faying surfaces of lap and straight tee joints shall be designed to be in intimate contact. A maximum opening of $\frac{1}{8}$ in. is permitted when assembled for welding.

13. *Strength Welds.*

(a) Do not guess at the strength of a weld, figure it.

(b) All strength welds are based on the thickness of the thinner material to be joined.

(c) Fillet welds in strength joints shall in general be standard $\frac{5}{16}$ and $\frac{3}{8}$ in. In general do not specify standard fillet welds smaller than $\frac{3}{16}$ in. nor larger than $\frac{1}{2}$ in.

(d) When intermittent welds are used, always weld the ends of the member on both sides.

(e) Location, width, and depth of reinforcement of butt welds shall be specified.

14. *Calk Welds.*

(a) Calk welds shall not be considered as contributing to the strength of a riveted joint.

(b) Fillet welds may be used for calking riveted joints and may be as small as $\frac{3}{16}$ in., provided the joint has not previously been subjected to working stresses. Mechanically calk and test a riveted joint in a pressure vessel

before applying a caulk weld. When joints with loose rivets are welded for tightness, strength welds shall be used.

(c) When welding is used for calking butt joints, a reinforced strength weld shall be used, unless special permission is obtained.

15. *Working Stresses.*

(a) Welded joints subject to bending and/or alternating stresses shall be designed to compel the maximum values of these stresses to take place in the base metal and not in the weld. This is possible by reinforcing the weld, the use of straps, etc.

(b) When lap or tee joints are used and strength is essential, specify a continuous strength weld on both edges, or on one edge, and a light continuous or an intermittent weld on the other edge, so as to prevent a bending stress in the strength fillet weld.

(c) In general use 36,000 lb. per square inch for the ultimate tension, compression and shear strengths of the weld metal and 9,000 lb. for the working strength. All values to be figured through throat of weld. (See latest codes of the American Welding Society or regulating body.)

16. *Warping and Its Prevention.*

(a) Metal arc welding will produce less warping than carbon arc welding.

(b) When metal arc welded lap or tee joints are employed and the assembled structure is inherently rigid prior to welding, warping will in general be negligible.

(c) When warping is likely to occur, use one or more of the following methods: Reduce the size of the welds to a minimum; use intermittent welds instead of continuous welds; jig, dog down, or employ additional stiffening members; keep work cool; use smallest electrodes and lowest current values which are suitable, and peen the weld.

17. *Size and Spacing of Assembly Bolts and Holes.*

(a) Assembly bolts shall in general be the same size as rivets of a corresponding riveted joint. Use the same size throughout the structure if possible.

(b) Assembly holes shall be of the size shown in the tables specifying these sizes.

(c) Assembly bolts, when necessary, shall not in general be spaced closer than 15 in. center to center.

(d) On work to be assembled in the field, consideration shall be given to providing a suitable number of assembly bolts for location purposes.

18. *Methods of Showing Welds on Drawings.*

The location, size and continuity of welds shall always be shown on a drawing. Do not merely specify "weld," as the design department is responsible for the correct size of all welds. The causes of weld failures in the past can very largely be attributed to poor design and the fact that the size and continuity of the weld was left to the judgment of the welder.

19. *Shop Requirements.*

Much of the foregoing requirements are applicable to the shop. In addition, the following shall be adhered to:

(a) No welder shall be permitted to weld important strength joints unless he has satisfactorily passed required tests to show his ability to make sound welds in the positions in which he will be called upon to weld.

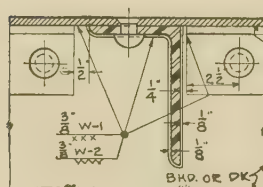
(b) Use maximum current values and largest electrodes suitable for the work, always being governed by such standards as have been provided.

(c) On all joints welded, the supervisor shall instruct the welder as to the electrode sizes and current values to be used. He shall also observe the ammeter on each operator's equipment to be assured that instructions are being followed.

(d) Each welder shall be furnished with a weld gage and the supervisor shall see that specified sizes of fillet welds and reinforcement of butt welds are being made.

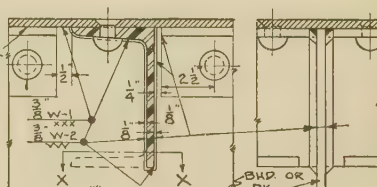
Section 6

TENTATIVE ARC-WELDING STANDARDS FOR SHIP'S HULL AND MACHINERY



ANGLE

(USED ON S.S. CALIFORNIA & U.S.C.G. CUTTER NORTHLAND)

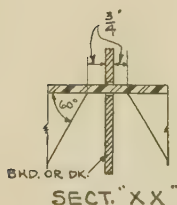


CHANNEL WITH BOTTOM FLANGE CUT.

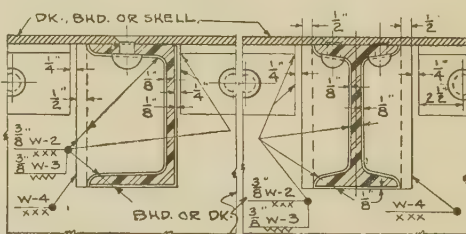
B (USED ON S.S. CALIFORNIA)

USE WHEN SHEARING AND BENDING STRESSES AT BHD. OR DECK DO NOT REQUIRE THE FULL SECTION OF MEMBER.

MEMBERS IN SLOTS.



SECT. "XX"



CHANNEL

I-BEAM

C

D

LAPPED BOSOM PLATES.

(PREFERRED)

USE WHEN SHEARING & BENDING STRESSES REQUIRE THE FULL SECTION OF MEMBER, (USED ON S.S. CALIFORNIA)

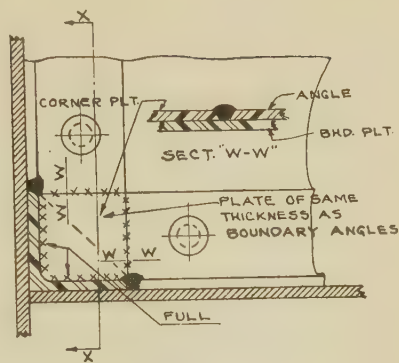
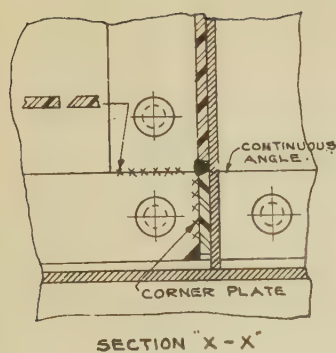
WELDING NOTES:

- W-1. COMPLETE THIS WELD FIRST.
- W-2. NEAR SIDE FOR WATER-TIGHTNESS AND NORMAL STRENGTH.
- W-3. BOTH SIDES FOR OIL & WATER-TIGHTNESS UNDER PRESSURE ALSO MAXIMUM STRENGTH.
- W-4. 1/4 WELD ON THIS EDGE WHERE DUST & RAT PROOF CONSTRUCTION IS REQUIRED.

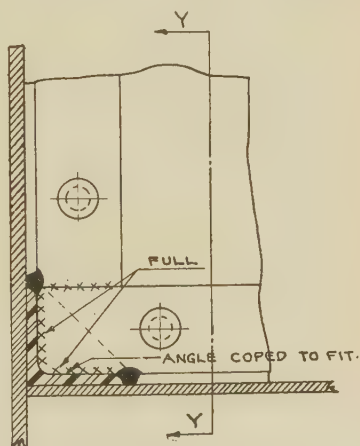
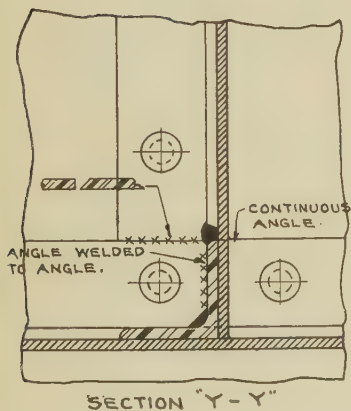
GENERAL NOTES:

1. ALLOW AN OVERALL CLEARANCE OF 1/4" ON SLOTS IN PLATING OF BHDs. & BHDs.
2. ALLOW 1/8" CLEARANCE ON BOSOM PLATES.
3. BOSOM PLATES ARE SAME WEIGHT AS BHD. & DK. PLATING.
4. STOPWATERS & OIL STOPS MAY BE FITTED & CONTINUOUS MEMBER CAULKED FOR 2" EACH SIDE OF BHD.

FIG. 25.—Welded connections of continuous members through bulkheads and decks, composite and welded construction.



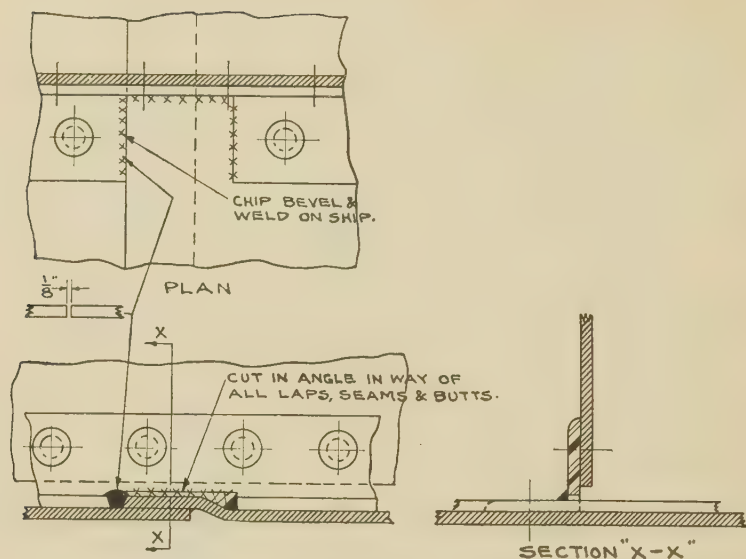
A. Corner formed with plate (preferred)



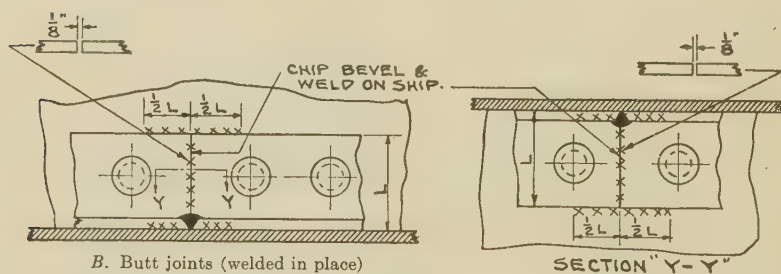
B. Corner formed by coping

FIG. 26.—Types of welded corners for watertight and oiltight bulkheads, floors, etc., composite and welded construction.

NOTE: Approved by American Bureau of Shipping for S.S. *Virginia*.



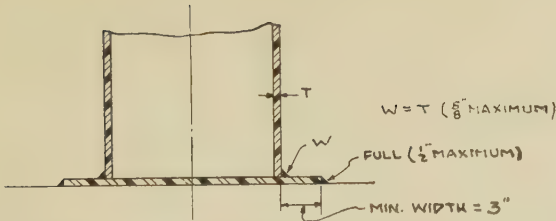
A. Detail of welded slot in oiltight, watertight, and non-watertight angles in way of seams and butts of plating



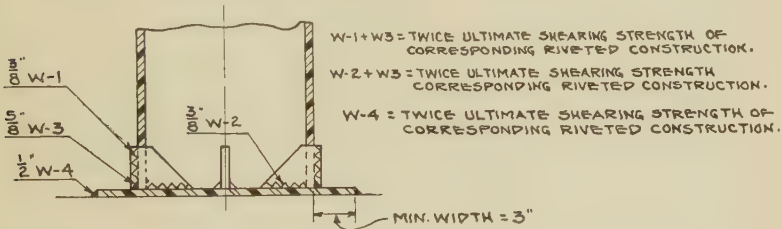
B. Butt joints (welded in place)

FIG. 27.—Welded boundary angle connections on ship, composite and welded construction.

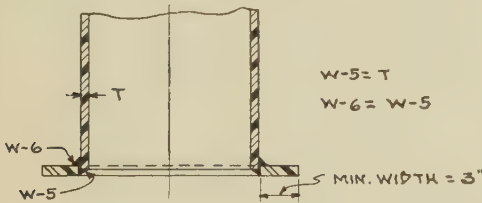
NOTE: (1) All fillet welds full; (2) approved by American Bureau of Shipping for S.S. Virginia.



A. Equivalent to or more than ultimate shearing strength of riveted construction (below 16 in. diameter).



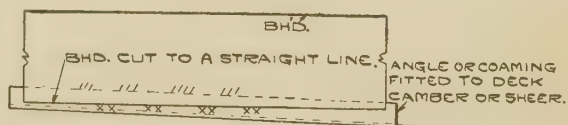
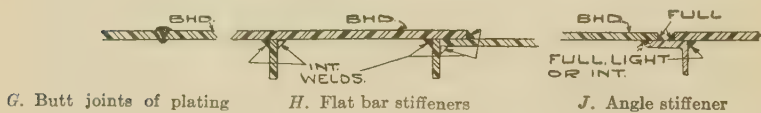
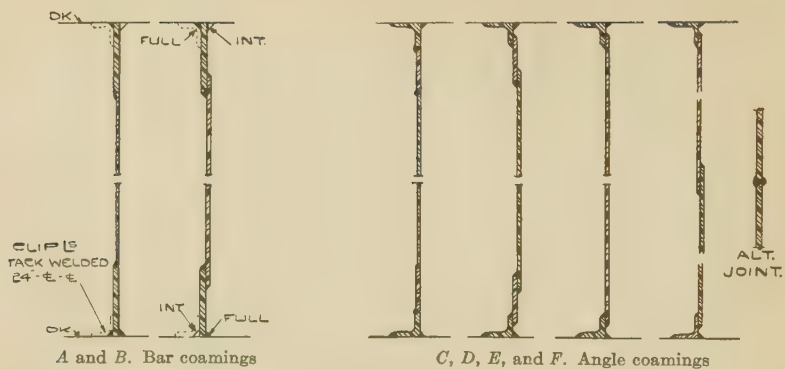
B. Equivalent to twice shearing strength of riveted construction (16 to 24 in. diameter)



C. For maximum strength and lightness (all diameters)

FIG. 28.—Welded pillars, all sizes.

NOTE: Detail "B" approved by American Bureau of Shipping for S.S. *Virginia*.



K. Method of fitting lap joints of coamings

FIG. 29.—Welded miscellaneous bulkheads.

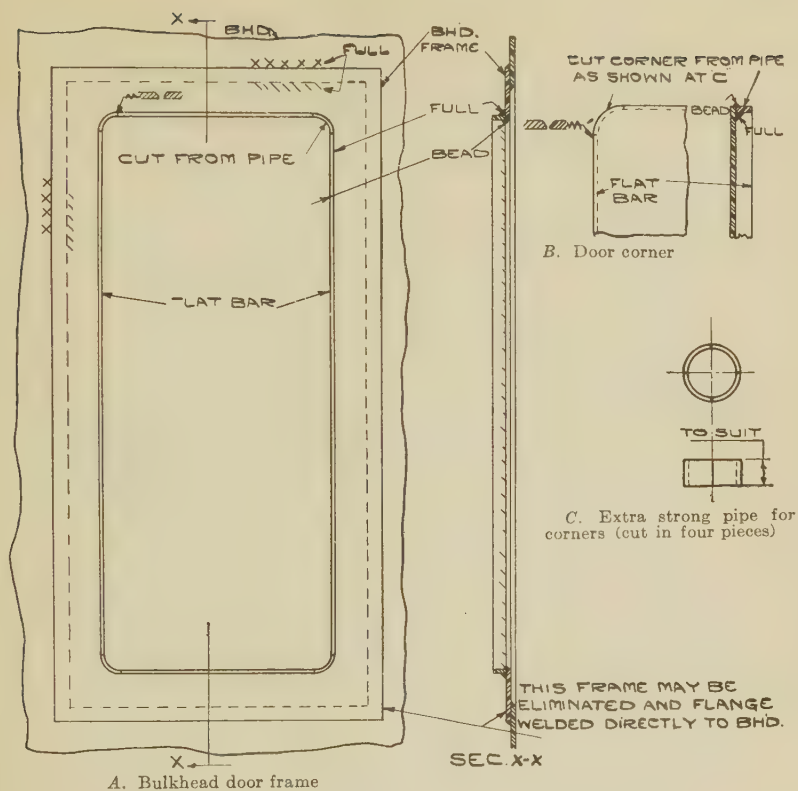
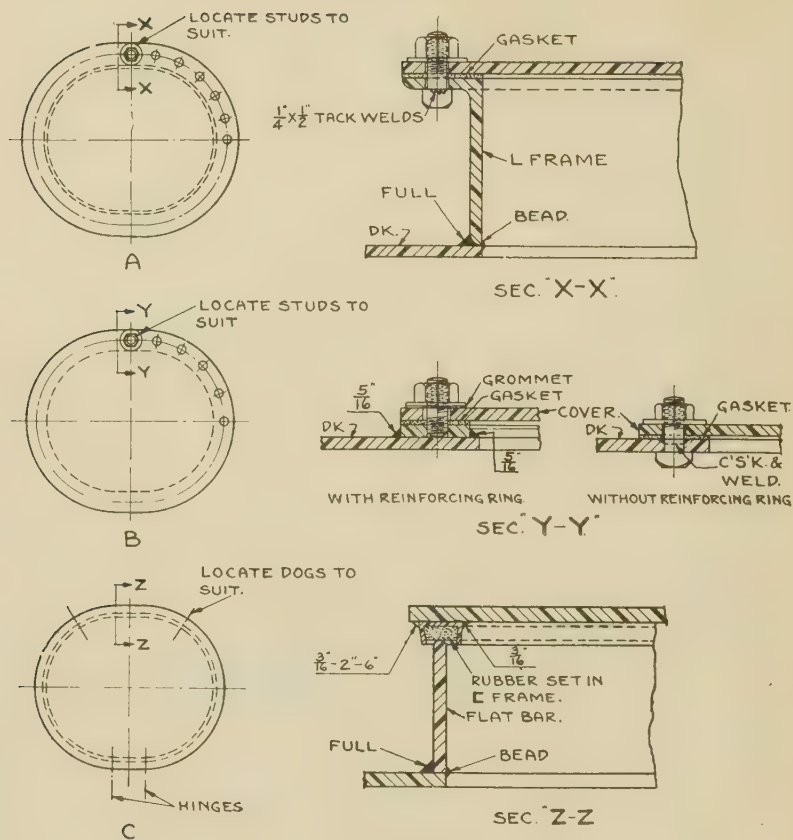


FIG. 30.—Welded watertight door and door frame.



NOTE—USED ON SEVERAL SHIPS.

FIG. 31.—Welded manhole coamings and covers.

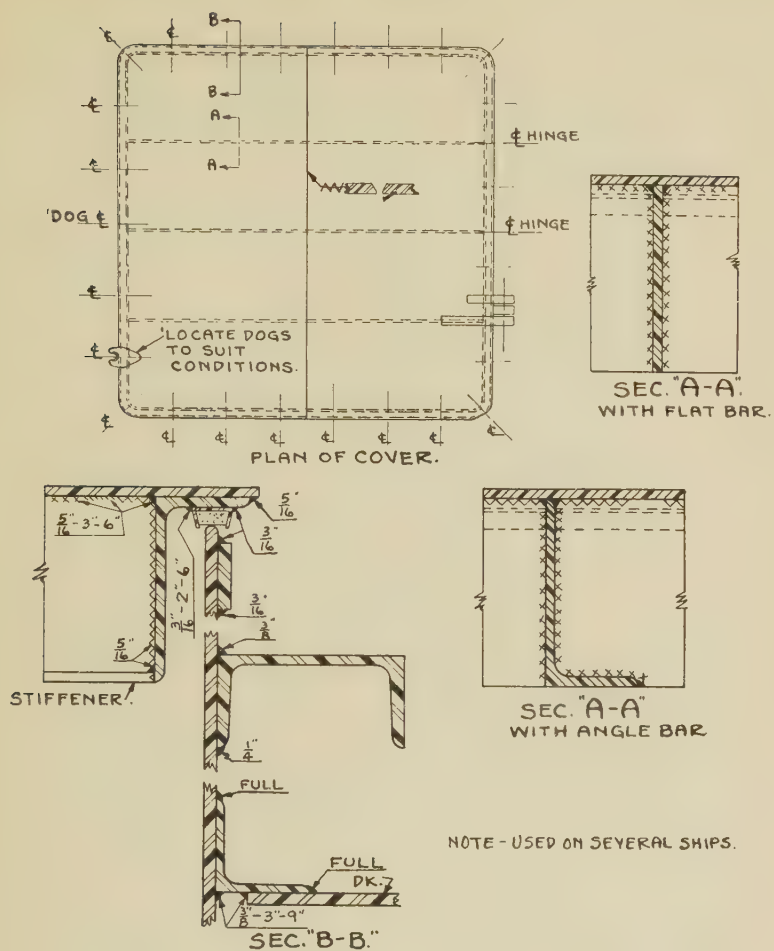
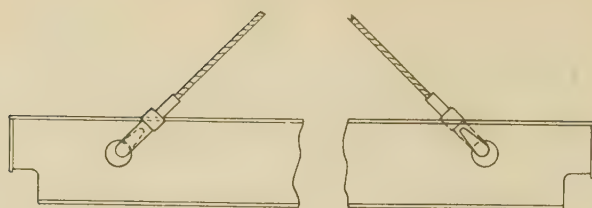
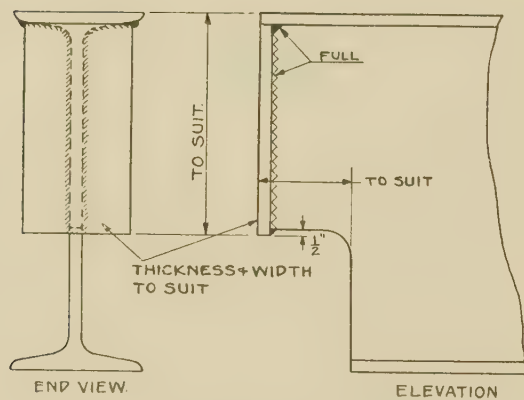


FIG. 32.—Welded coaming and cover for cargo and companion hatches.



LIFTING SLING.



END VIEW.

ELEVATION

FIG. 33.—Welded cargo hatch web beam.

NOTE: Similar design approved by American Bureau of Shipping for S.S. *Caracas*.

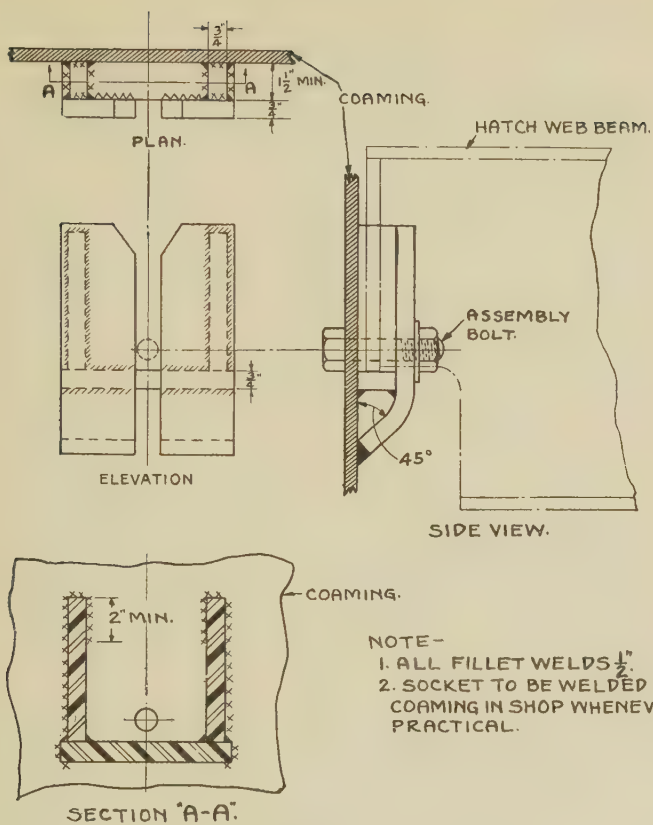


FIG. 34.—Welded hatch beam socket.

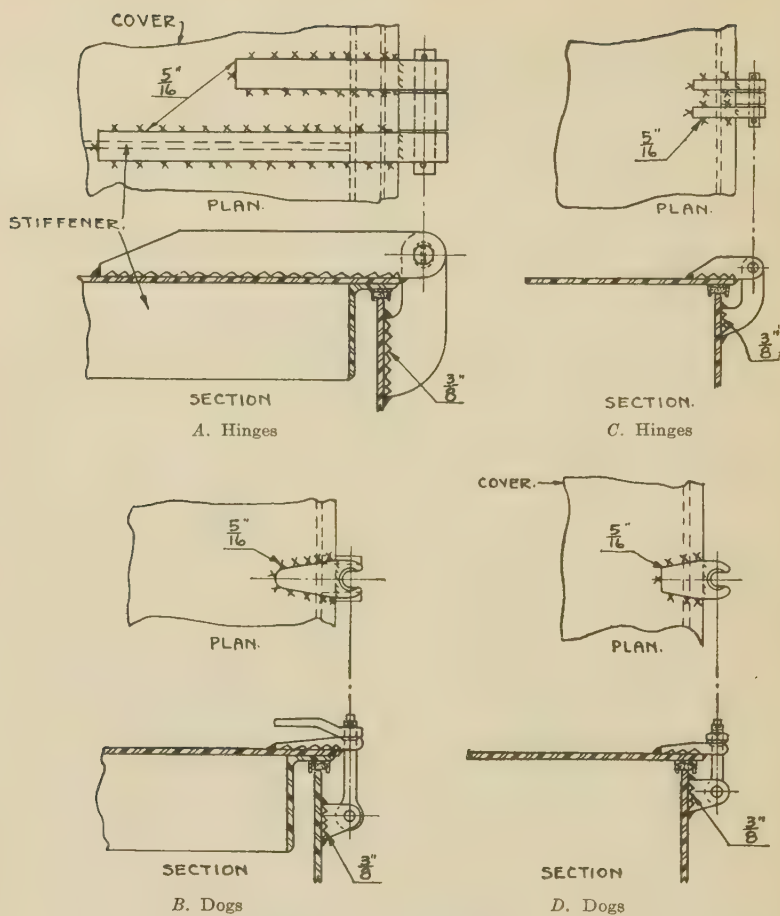
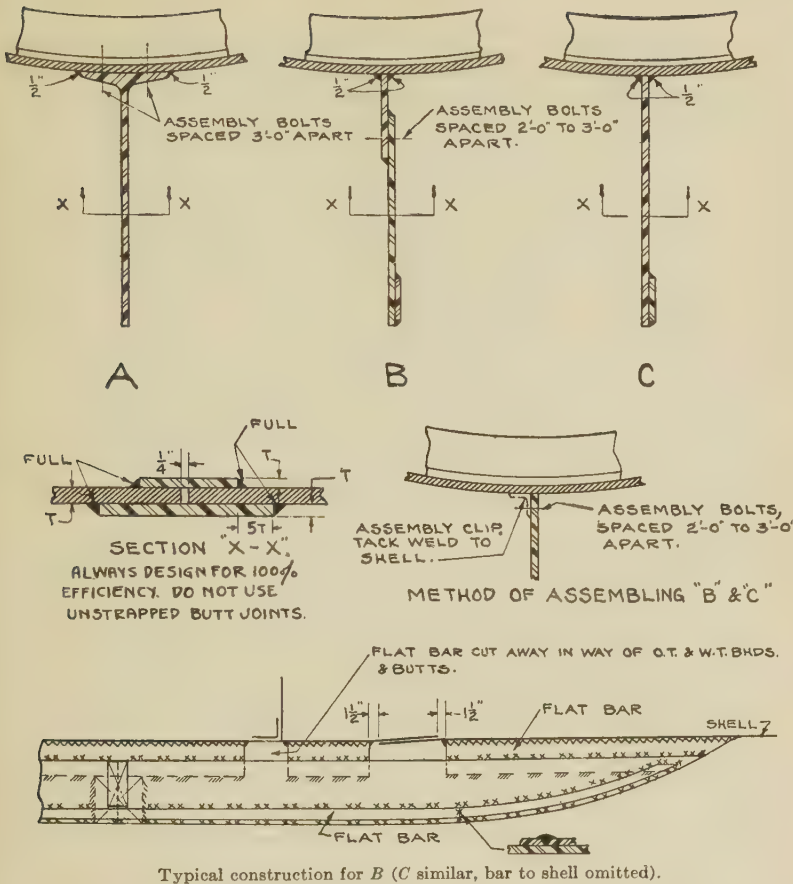


FIG. 35.—Welded watertight cargo hatch fittings and manhole fittings.

NOTE: Used on several ships.



Typical construction for B (C similar, bar to shell omitted).

FIG. 36.—Types of bilge keels, composite and welded construction.

NOTE: (1) Detail C used on four colliers of the U. S. Navy; (2) special care shall be used to obtain butts of 100 per cent efficiency; (3) Detail B approved for S.S. *California* by American Bureau of Shipping.

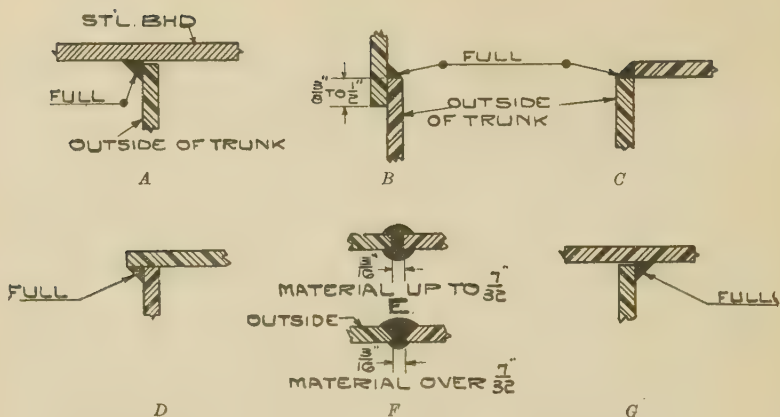
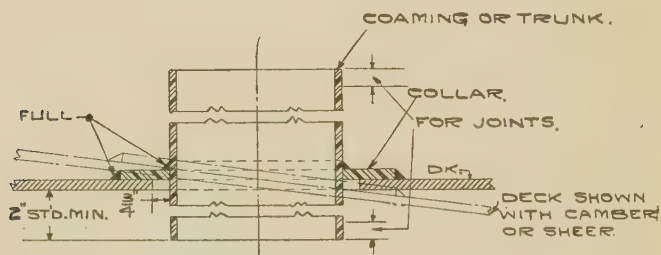
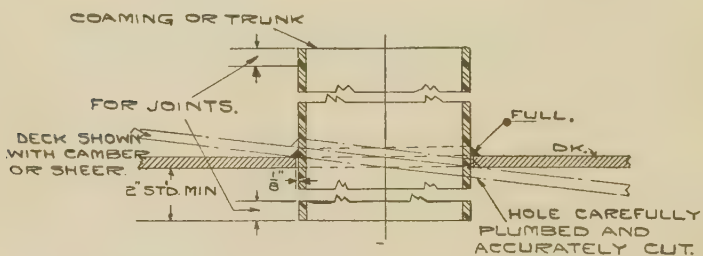


FIG. 37.—Details of joints for welded vent ducts and trunks.

NOTE: Approved by American Bureau of Shipping for S.S. *California*, *Virginia*, and other ships.

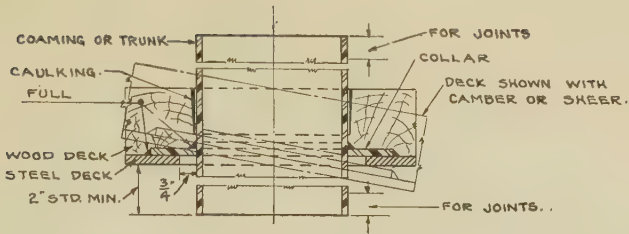


A. Flat collar attachment (preferred) (approved by American Bureau of Shipping for S.S. *Virginia*)

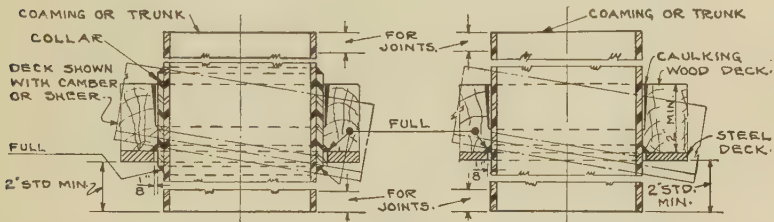


B. Deck attachment without collar (special) (use to secure maximum saving in weight)

FIG. 38.—Welded vent coaming and trunk attachments for steel decks.



A. Flat collar attachment (preferred) (use where weight of coaming material permits calking)



B. Reinforcing collar used to permit calking of light materials (special)

C. Use to secure maximum saving in weight when weight of material permits calking (special)

B and C. Deck attachments without angle and flat collars

FIG. 39.—Welded vent coaming and trunk attachments for calked wood decks.

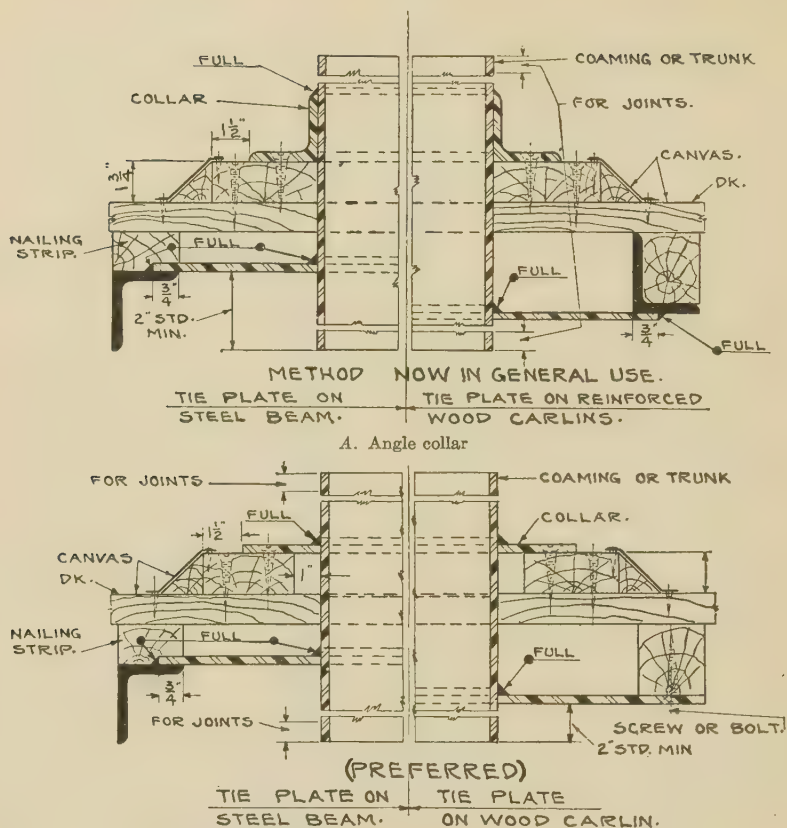


FIG. 40.—Welded vent coaming and trunk attachments for joiner deck.

NOTE: Approved by American Bureau of Shipping for S.S. *California*, *Virginia*, and other ships.

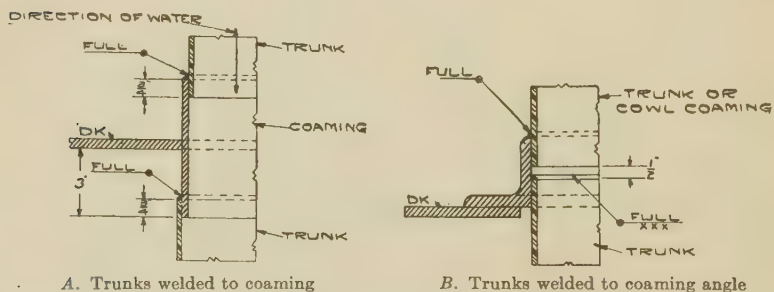


FIG. 41.—Welded joints of ducts and trunks to coamings.

NOTE: Approved by American Bureau of Shipping for S.S. *California* and *Virginia*.

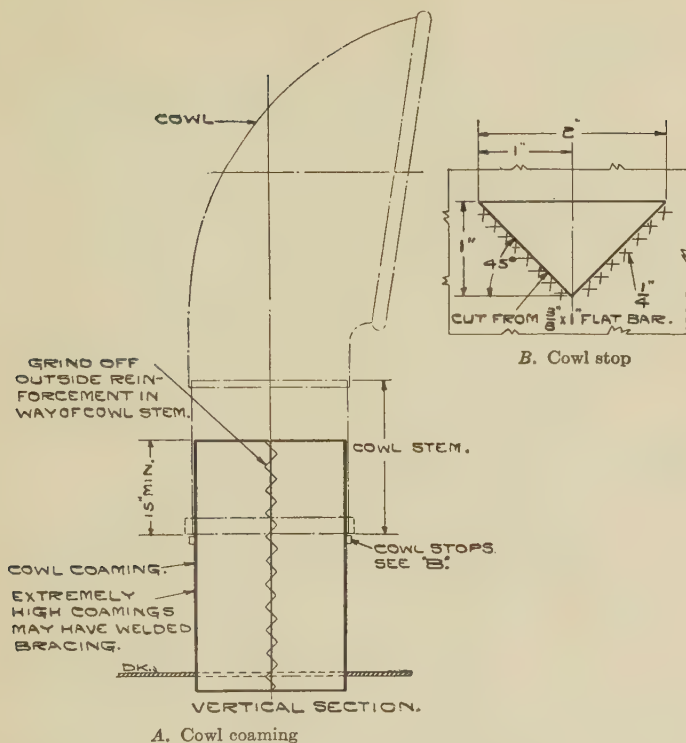


FIG. 42.—Welded coaming details for vent cowl.

NOTE: Approved by American Bureau of Shipping for S.S. *California*, *Virginia*, and other ships.

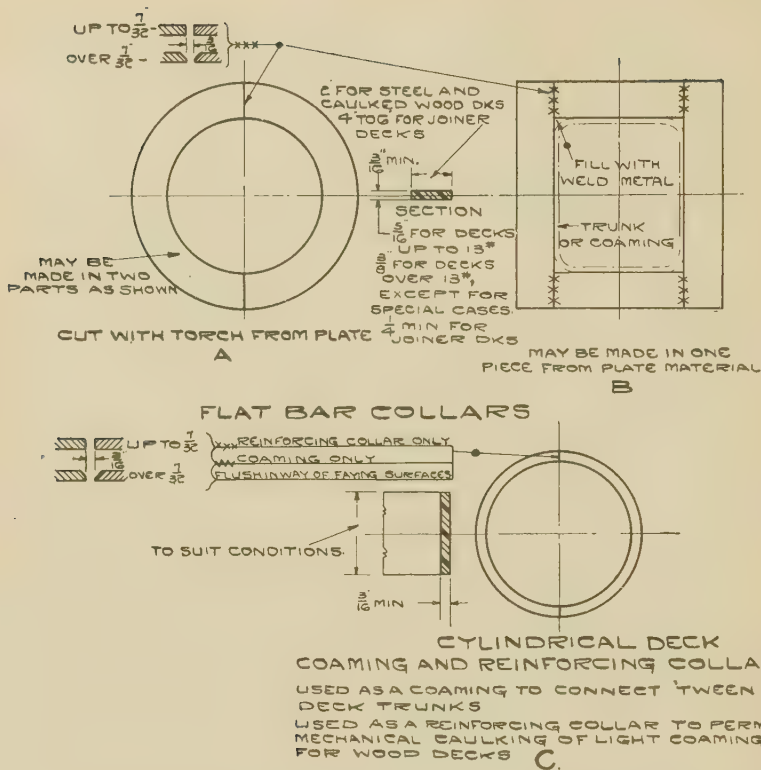
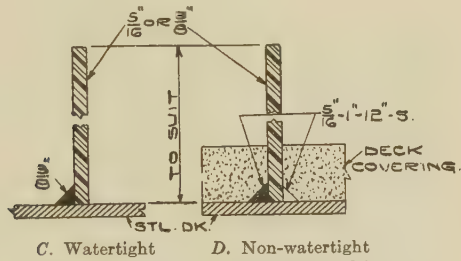
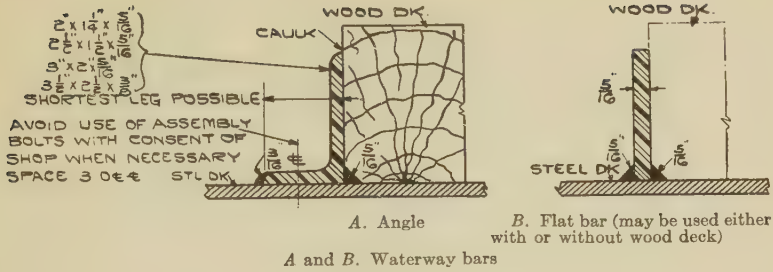


FIG. 43.—Welded coaming collars for vent ducts and trunks.

NOTE: (1) When the camber and sheer of a deck is great, split ring in two parts, place in position on the deck, and weld the resulting crack as a butt joint; (2) approved by American Bureau of Shipping for S.S. *California*, *Virginia*, and other ships.



C and D. Miscellaneous coamings (for shower baths, foundation drip pans, scuttle butt drip pans, etc.)

FIG. 44.—Welded waterway bars and miscellaneous coamings.

NOTE: Used on several ships.

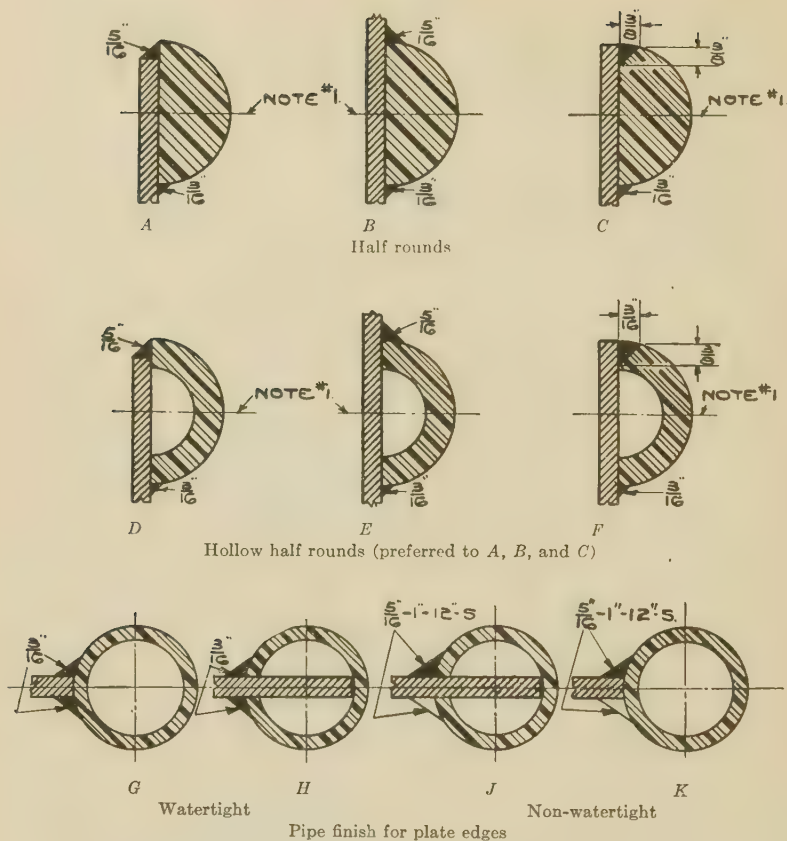


FIG. 45.—Welded half rounds and split pipe.

NOTE: (1) Avoid, if possible, the use of assembly bolts—when necessary, space 3 ft. center to center; (2) detail C used on S.S. *California* and U. S. Coast Guard cutter *Northland*.

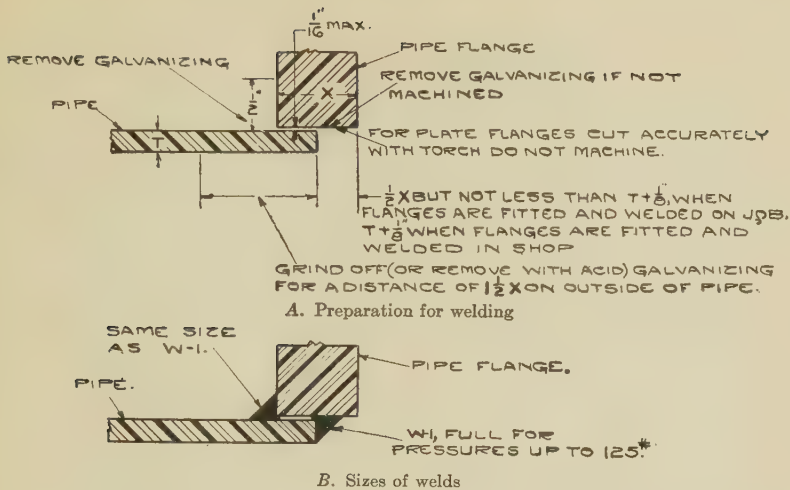


FIG. 46.—Welded black and galvanized pipe flanges for low-pressure work.
NOTE: Used on several ships.

Section 7

LIST OF ARC-WELDING APPLICATIONS TO SHIPS' HULLS AND MACHINERY BY THE NEWPORT NEWS SHIPBUILDING AND DRY DOCK COMPANY

NOTE.—Unless otherwise stated, applications listed have been used on one or more ships. Some of the most important of the applications have been referred to in the text.

ARC-WELDING APPLICATIONS TO SHIPS' HULLS

Part of vessel	Amount of welding ¹	Part of vessel	Amount of welding
A-frame to deck.....	B	Bilge keel half-rounds.....	A
Angle and channel butts...	A	Bleeder plugs.....	A
Beams to bulkhead with bosom plate.....	A	Bulkheads, N. W. T.....	A
Beams to bulkhead without bosom plate.....	A	Bulkheads, miscellaneous.	A
Belaying-pin rail.....	A	Bulkheads, fire screen....	B
Bilge keel complete to shell.....	A	Bulkheads, corrugated....	B
		Bulkheads, expanded metal.....	B
		Bulkheads, toilet.....	B

¹ Indications: A—Completely welded. B—Main portion welded. C—Incidental welding. (a)—Alternate beams to. (b)—Proposed. (c)—Corners only.

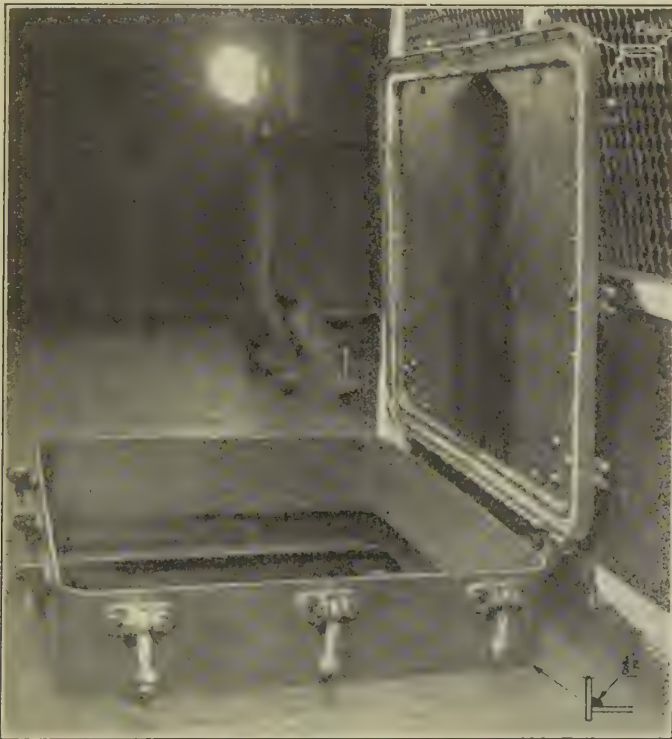
Part of vessel	Amount of welding	Part of vessel	Amount of welding
Cargo port brows.....	A	Half-rounds and ovals.....	A
Cargo port door.....	A	Ventilation, natural.....	A
Cargo port, frame.....	C(e)	Waterway bars.....	A
Cargo batten bar.....	A	Hatch, cargo, parts of.....	B
Casings, engine.....	B	Hatch, companion.....	A
Casings, motor.....	B	Hatch, web beams.....	A
Chain pipe, seams of.....	A	Ladder fastenings.....	A
Chain plates.....	C	Lashing rings.....	A
Chain locker division bulk- head.....	A	Lito silo clips.....	A
Clips, angle and bar.....	A	Manholes.....	A
Coamings, miscellaneous bulkhead.....	A	Mast fittings.....	A
Collar at bulkheads and decks.....	A	Mast fittings, pads and butts of.....	C
Crow's nest.....	A	Mast complete.....	A(b)
Deck house, superstructure	B	Pads, lifting rudder, etc...	B
Deck plating.....	B	Pillars, pipe, and H-section	A
Deck rail.....	A	Pipe tunnel.....	A
Doors, W.T. and N.W.T., etc.....	A	Reinforcing rings.....	A
Drain well.....	A	Room, lamp.....	B
Expanded metal, bulk- heads.....	B	Rudder.....	A
Fenders, butts of.....	A	Shell seam, calking of.....	A
Fenders, to shell.....	A(b)	Skylight and fittings.....	A
Fittings, hatch, etc.....	A	Slop chute.....	B
Foundations, main.....	C	Stapling, angles, etc.....	A
Foundations, miscellaneous	A	Steering quadrant.....	A
Galley, smoke pipe coam- ing.....	C	Stiffeners, miscellaneous bulkhead.....	A
Girders, longitudinal.....	C(a)	Store and shop room fit- tings.....	A
Gratings, chain locker.....	B	Swimming tank.....	A
		Superstructure plating....	B
		Tanks, independent.....	A

ARC-WELDING APPLICATIONS TO SHIPS' MACHINERY

Part of vessel	Amount of welding	Part of vessel	Amount of welding
Braces, boiler to F. O. bulkhead.....	B	Ducts, forced-draft.....	B
Bracket pads.....	B	Electrical boxes.....	A
Coils, oil, tank-heating....	A	Exhaust trunk, main.....	A
Condensers.....	A	Gratings, engine-room....	B
Cowls.....	A	Gratings, boiler-room....	B
Drain wells.....	A	Hangers, pipe.....	A
		Hangers, wiring.....	A

Part of vessel	Amount of welding	Part of vessel	Amount of welding
Housing and guards.....	A	Smoke stack, details.....	C
Pipe, air intake, 14-in. diameter.....	B	Stack, vent details.....	C
Pipe branches.....	A	Tanks, independent.....	A
Pipe flanges, to pipe, deck, and bulkhead.....	A	Tanks, floats in.....	B
Piping, supports of.....	A	Trough, wash crews.....	A
Pipe, water cool, exhaust..	B	Vent, Mechan. heavy cargo.....	A
Shaft guards.....	A	Vent, Mechan. light Gal- vanized.....	C
Smoke stack, donkey boiler	C		

Section 8

MISCELLANEOUS ILLUSTRATIONS OF ARC-WELDING
APPLICATIONSFIG. 47.—Arc-welded hatch coaming and cover, Coast Guard cutter *Northland*.

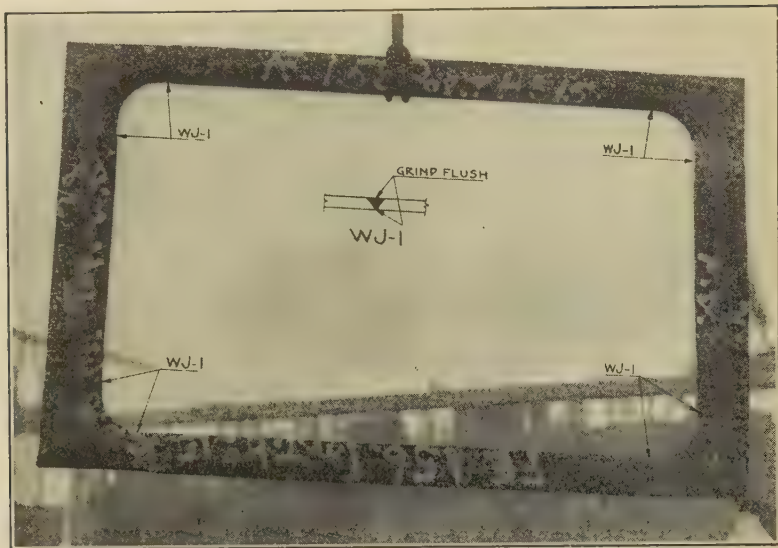


FIG. 48.—Arc-welded cargo port frame.

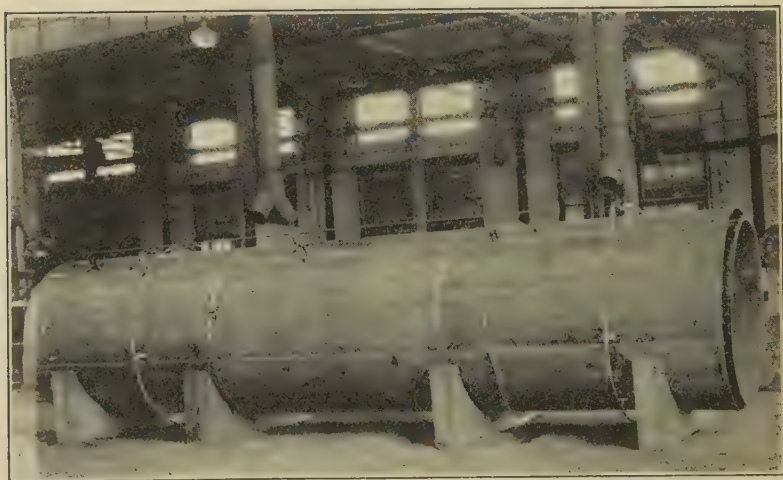


FIG. 49.—Arc-welded destroyer smokestack.

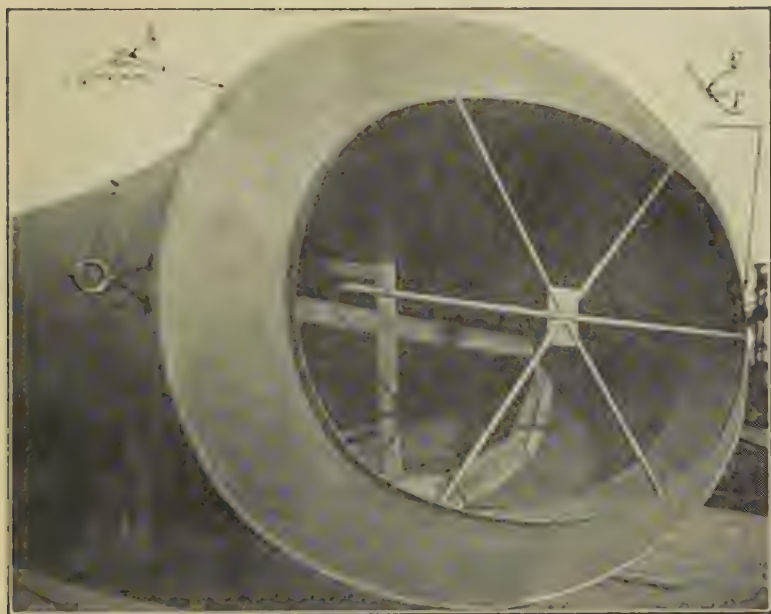


FIG. 50.—Smokestack, composite construction.

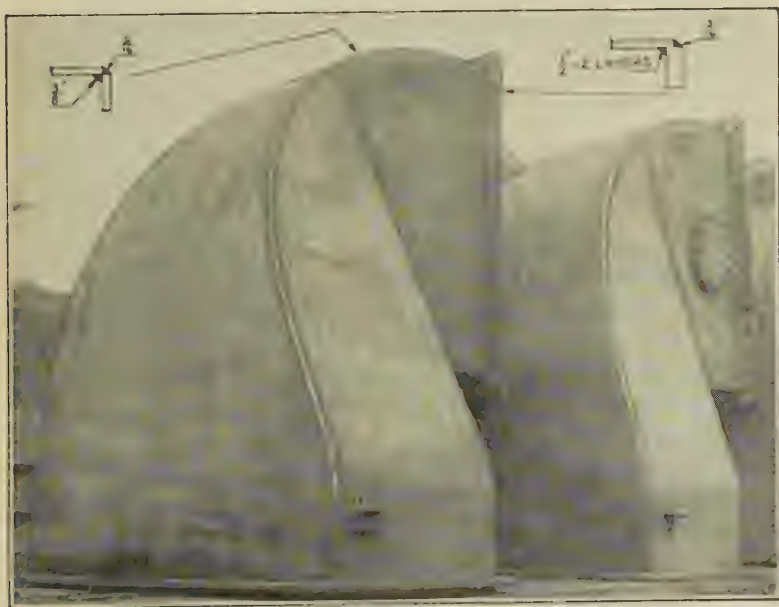


FIG. 51.—Arc-welded exhaust trunks for steam turbines.
No assembly holes used



FIG. 52.—Arc-welded intake duct for scavenging air for Diesel engines.



FIG. 53.—Arc-welded vent coaming for joinder deck (see Fig. 39 B).



FIG. 54.—Arc-welded vent ducts in engine room.

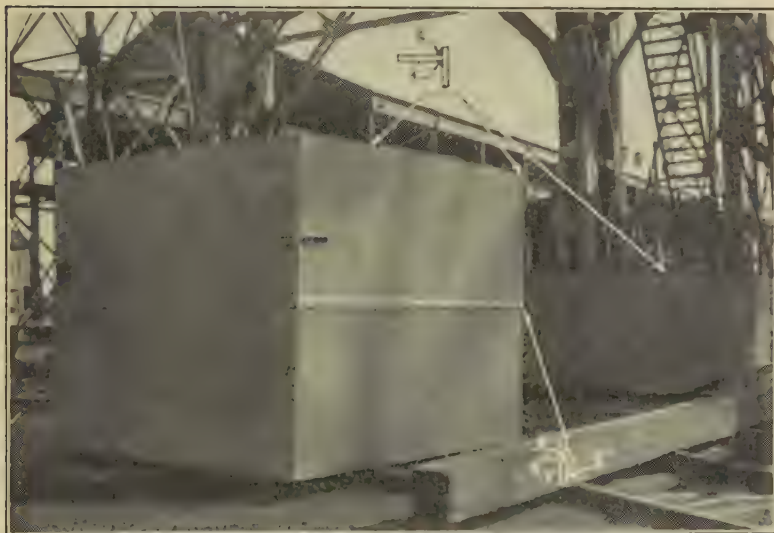


FIG. 55.—Arc-welded fresh-water tank.
No assembly holes used.



FIG. 56.—Arc-welded extension to foundry cleaning room, Newport News Shipbuilding and Dry Dock Company.

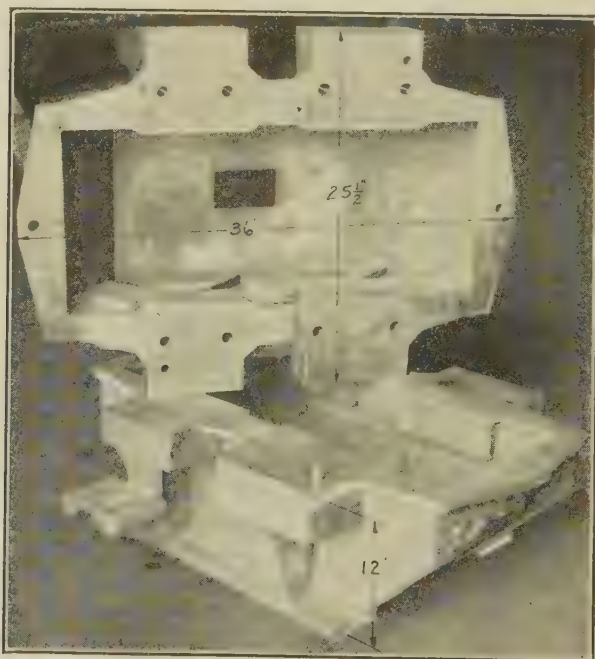


FIG. 57.—Arc-welded gear housing for 10-ton crane.

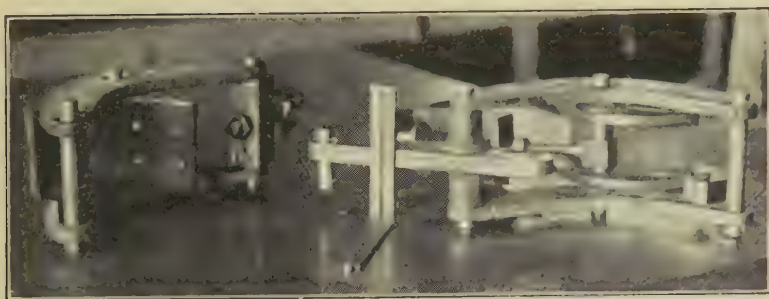


FIG. 58.—Arc-welded brake shoe for 10-ton crane.

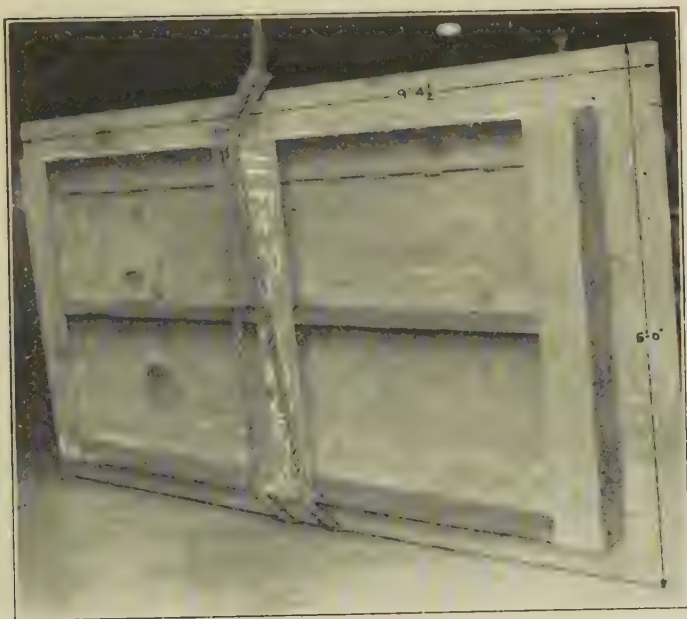


FIG. 59.—Arc-welded face plate, replacing a casting.

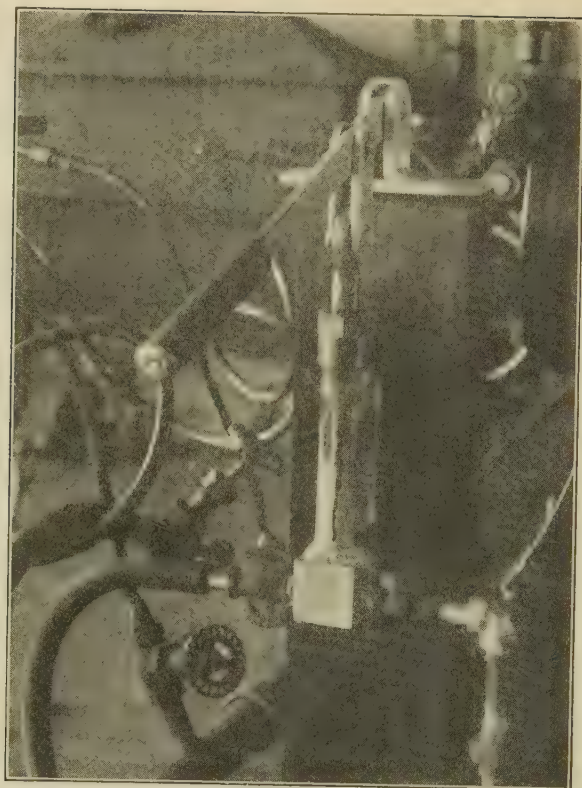


FIG. 60.—Arc-welded acetone pump for gas plant.



FIG. 61.—Welded trash racks.

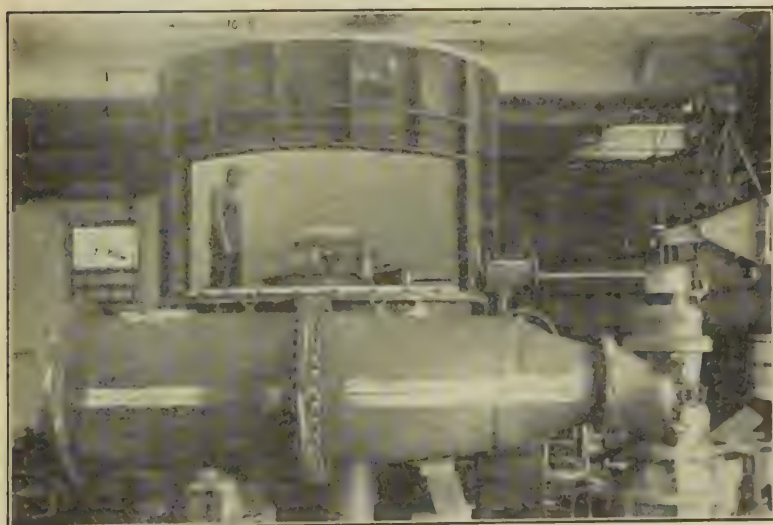


FIG. 62.—Arc-welded generator support for 10,000-hp. hydraulic turbine.

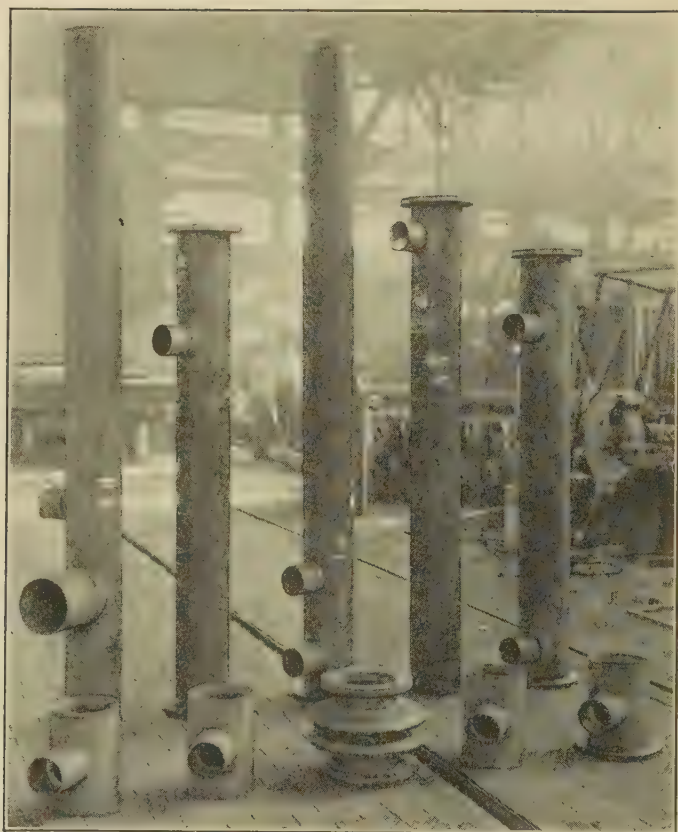


FIG. 63.—Arc-welded oil piping with branches.

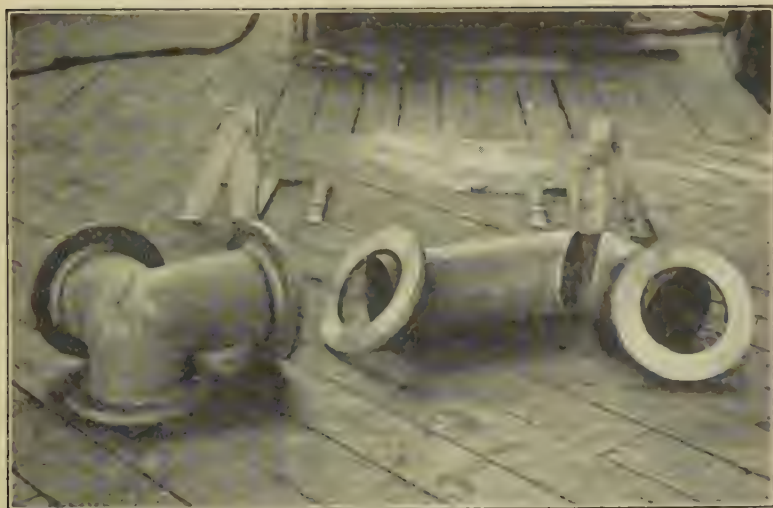


FIG. 64.—Arc-welded side outlet elbow.
Flanges and pipe cut with gas torch.



FIG. 65.—Arc-welded material skip.



FIG. 66.—Arc-welded 100,000-gal. swimming pool.

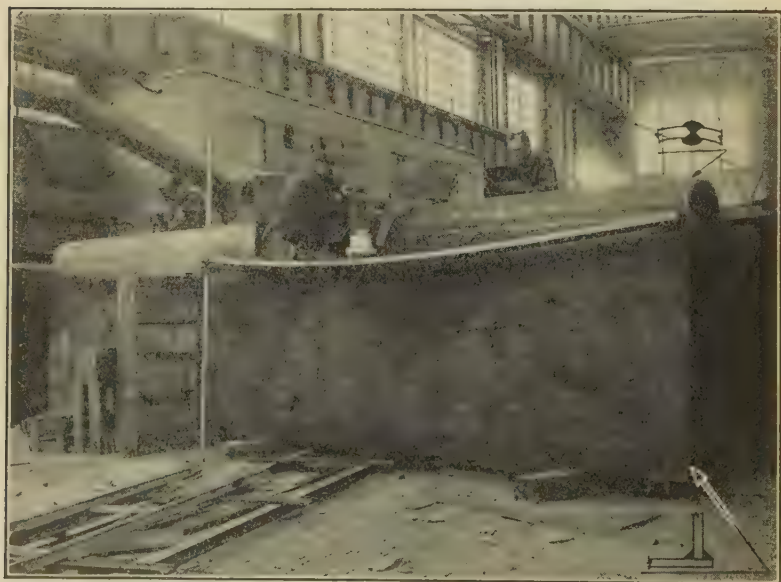


FIG. 67.—Arc-welded galvanizing tank.

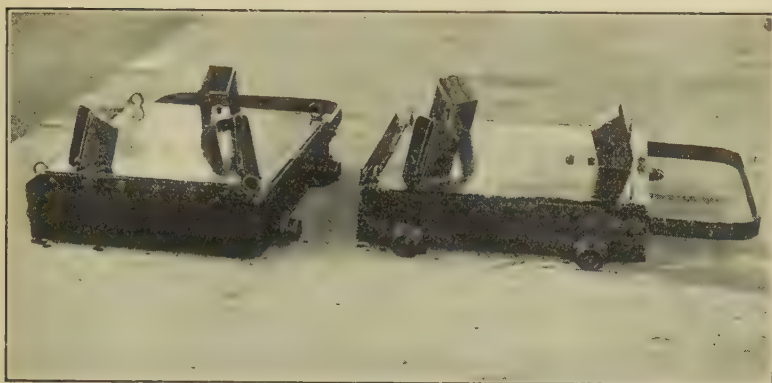


FIG. 68.—Arc-welded mine trucks.



FIG. 69.—Filling blow holes in an iron casting by the carbon arc.

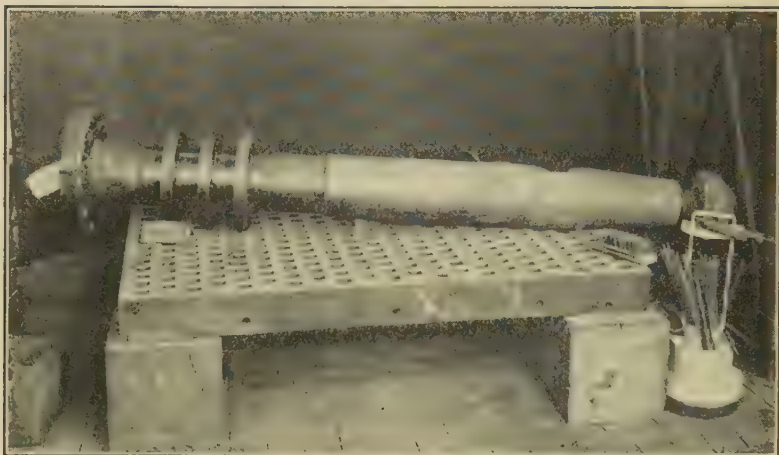


FIG. 70.—Worn shaft being built up with the metal arc.



FIG. 71.—Building up a piston with the metal arc.



FIG. 72.—Saw teeth and pistons built up with the metal arc.



FIG. 73.—Worn non-slip built up with the metal arc.

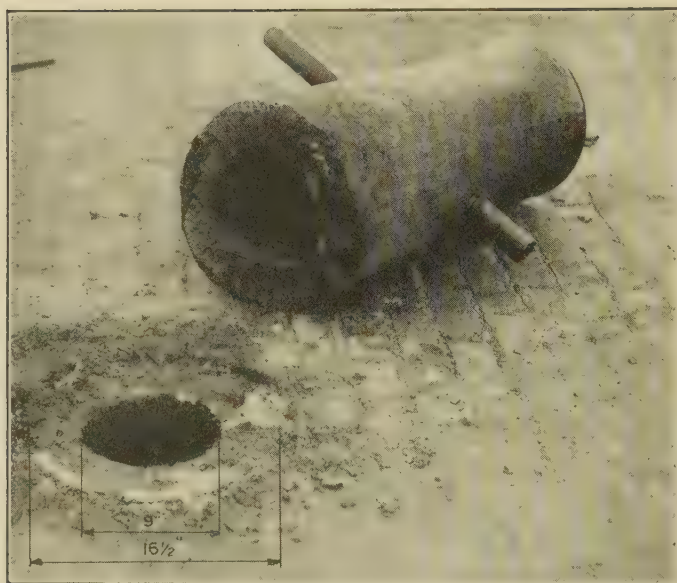


FIG. 74.—Cast-iron cannon cut with the carbon arc.

PART II
FUNDAMENTAL PRINCIPLES OF ARC WELDING
BY
H. DUSTIN

INTRODUCTION

Papers have been requested disclosing the advancement in the art of welding. It seemed to the author that a summary of the results of the experimental researches on arc welding made in his laboratory since 1925 fulfilled these requirements. The aim of these researches was to ascertain rules for a rational calculation of welded joints, with particular reference to structural work.

This research, now practically completed, has enabled the framing of precise and simple rules, which have been tested by a number of experiments. By these rules it is possible to calculate without any difficulty welded structures in which all the parts have the same factor of security, but in which the whole of the metal is used to its full strength. In other words, these rules make it possible to calculate rapidly, directly, and very simply the most economical structure that can be achieved with a prescribed factor of security.

There are certain accepted opinions with regard to welded structures, and these tests have shown how little justification there was for most of these. The practical man certainly will be surprised at the smallness of the welds sufficing to insure a given strength. If the methods of calculation which are suggested were applied to most of the existing structures, it would be found that much current, welding metal, and labor had been wasted. These experimental researches have shown:

1. How to obtain the greatest economy reasonably possible in steel structures.
2. The saving that may be made in each welded joint; when one thinks of the number of welds made year in and year out in structural work, one realizes what economy this means.
3. The real behavior of welded structures under dynamic stresses, repeated, and secondary stresses, and their superiority in this respect over riveted structures.

CHAPTER I

PRELIMINARIES OF TESTS IN 1925-1926

Toward the end of 1924 the author was asked by friends of the S. A. Soudure Electrique Autogène, Brussels (*A.R.C.O.S.*), to carry out systematic experiments with a view to determining the experimental bases of a rational calculation of welded joints, with particular reference to structural work.

In order to make a rational study of welded joints, it was decided first of all to acquire more complete and reliable information about the metal of the weld itself and about the possible action of the weld on the metal of the welded parts. The following sums up what was known about this subject at the end of 1924:

A. The Metal of the Weld.—The ultimate tensile strength (Z) was given by a lot of tests, and the Brinell hardness also (Δ).

There was very little information regarding the elastic limit in tension (L). As to the corresponding elastic modulus (E), Young's modulus, the Lloyd's had just published results of tests with surprising figures like 16,500 kg. per square millimeter.¹

Very few tests had been carried out as regards resistance under shear-stress, a kind of stress that occurs very frequently in structures.

With regard to impact tests, contradictory results had been obtained. The general opinion was that the metal of the weld was rather brittle and little resilient; nevertheless, results from excellent tests were published. Most of these tests were carried out with non-standard specimens, using non-standard methods. Further experiments were necessary before a well-founded opinion could be formed.

Little confidence was placed in the resistance of this material under repeated stresses. For instance, in the fatigue test by the rotating-beam method, the Lloyd's required only that the test specimens should hold out during 5,000,000 cycles under a tension of 9 kg. per square millimeter. This seems very little for a metal that normally has an ultimate tensile strength of approximately 40 kg. per square millimeter, often reaching 50 kg.

¹ The metal of the weld is mild steel; Young's modulus for steels varies between very close limits, never under 20,000.

B. Action of the Weld on the Metal of the Welded Parts.—It was generally reported in the shops that the weld had a detrimental effect upon the adjacent metal, resulting in a decided weakening.

It had been observed that when a welded joint between plates or shapes of ordinary commercial quality was tested up to breaking point, the fracture occurred generally not in the weld itself but frequently just beside it.

No reasonable explanation of this phenomenon was given; it seemed, however, very important to ascertain whether there really was such a weakening of the metal and to what it must be attributed.

The work of this experimental campaign of 1925–1926 was devoted to these preliminary questions. The investigations were confined strictly to electric-arc welding of structural steels; *i.e.*, to the arc welding of mild or medium steels.

Already at the end of 1924 welding rods made of bare wire were considered by the best European specialists as obsolete and inadequate for modern technical requirements; for these experiments only coated electrodes of unquestioned quality were used. First, tests were made of electrodes of fourteen different brands, for the mechanical qualities and soundness of the deposited metal, and three of these were retained for the systematic work. These three were chosen not because they were the best but because they were characteristic types of three different categories.

All three had a core of mild steel wire of good average quality. The difference between them was in the coating. The first (referred to under initial Sp.B.) was electrolytically nickel plated (1.25 per cent Ni) and then further heavily coated, this coating being held together by coiled asbestos wire. The second (initial T) was merely thickly coated without any special support. The third (N) had only a thin coating. The composition of the coatings is kept secret by the makers; analyses of various coatings have been published in technical papers (such as *Revue de Métallurgie*, June, 1925).

The first (Sp.B.) is a high-grade electrode which deposits a perfectly clean metal with a resistance up to 50 kg. per square millimeter. The second (T) is a fair average specimen of good coated electrodes such as are in use in various countries advanced in the art of welding. It deposits a very clean metal with a resistance up to 40 kg. per square millimeter. The third (N)

also deposits a metal with a resistance of 40 kg. per square millimeter, but not so clean nor so even.

The first two (Sp.B. and T) produce a profuse, thick slag and are specially suitable for forming resisting joints. The third (N) produces a light, fluid slag that makes quick work possible and therefore is suitable rather for light or non-resisting welds. The metal deposited by all three is mild steel containing 0.10 to 0.15 per cent of carbon. T and N give a deposited metal very similar to structural steel; Sp.B. gives rather a light alloy steel.

With these electrodes there were prepared more than 300 test pieces of all shapes and sizes. More than 200 specimens also were polished for metallographic examination.

The results of these preliminary tests and examinations are summarized in the memorandum published by the *Revue Universelle des Mines et de la Métallurgie*, December, 1926. The main conclusions from this memorandum are as follows:

1. The fact that stands out above all others is the great importance of the choice of the electrode.

2. The metal deposited by good coated electrodes behaves under static loads like good mild steel. The ultimate tensile strength (Z), the apparent limit in tension, or yield point (L), the elastic modulus (E), Young's modulus, also the characteristics in shear (Z'' , L'' , E'') are quite normal and, what is the most important, vary but slightly.

3. It follows that the dimensions of the welded joints can be calculated with reliable results.

4. . . .

5. The Charpy impact test is totally inadequate in itself to decide upon the brittleness or non-brittleness of the welds. Tests with dynamic loads ought to be carried out under conditions identical, so far as possible, with those that occur in practice. The behavior of the three electrodes under dynamic stresses was very different.

6. The fatigue tests should be continued, since lack of time prevented their completion. Such as were carried out by the rotating-beam method showed that for specimens without defect the endurance limit could be taken as equal to half the apparent elastic limit.¹

¹ The apparent elastic limit (or yield point) is 36 kg. per square millimeter for Sp.B. and 30 kg. for T. The corresponding "endurance limits" are 18 kg. and 15 kg. Fatigue tests were not carried out with N.

7. . . .

8. Welding does not in any way affect the metal welded—at least mild or medium steel. It acts like a very localized “normalizing” and has exactly the same effect as this well-known heat treatment.

9. There is nothing whatsoever to be gained by annealing the welds.

CHAPTER II

CONTINUATION OF FATIGUE TESTS IN 1927

It has been seen that for "faultless" weldings the endurance limit may be taken as equal to half the apparent limit of elasticity (yield point). The aim of the experiments in 1927 was to determine how much this figure must be reduced in practice to allow for small defects that must be looked for in a cast metal that has not undergone any mechanical working-up processes. All experiments were made on Swedish machines made by the Aktiebolaget Alpha, and for these the test specimens were of the American Farmer type. This specimen was found the most satisfactory from every point of view.

In 1926 the test specimens were built up as shown in Fig. 1 (this was done with the idea of reducing preparation work to a minimum). Two round bits of medium steel B_1 and B_2 , 20 mm.

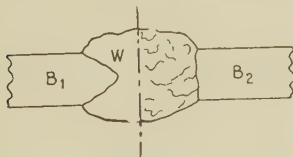


FIG. 1.



FIG. 2.

in diameter, were placed opposite each other on a plate; the welding block W was then made, and lastly the whole was brought to the lathe and machined to correct size and shape. These specimens contained defects—very slight but fairly numerous. A large proportion, more than half, broke after merely some ten thousand or hundred thousand revolutions. On the other hand, in order to save time, the machine was run at 3,000 r.p.m., at which speed vibrations, which affect the results, were frequently set up spontaneously.

In 1927 in order to eliminate these drawbacks the speed was reduced to 1,500 r.p.m., and the test specimens were prepared in the way recommended by the British Lloyd's. In a plate 20 mm. thick a groove $A-B-C$ was made on the shaping machine (see

Fig. 2) and the weld W_1 was made in this groove. The plate was then turned over, grooved again $D-E-F$, and in this groove the weld W_2 was made. The result was the same as if there had been made a wide, very sound X-shaped weld between the two halves of the plate. It is well known that such welds are usually very good. The plate was then cut into square bars 20×20 mm., which were brought to the lathe. This time the results obtained were remarkably uniform, as is shown by Table 1.

TABLE 1.—FATIGUE TESTS, 1927
(Electrode T used in systematic tests)

Outer fiber stress, kilograms per square centimeter ¹	Revolutions before fracture	Outer fiber stress, kilograms per square centimeter ¹	Revolutions before fracture
20	360,000	18	715,000
20	160,800	16	2,344,600
20	422,000	16	2,312,500
18	668,000	16	8,243,000
18	1,120,000	15	∞ ²

¹ Millimeter (?)—Ed.

² Six pieces unbroken after 10,000,000 to 15,000,000 revolutions.

From this Table 1 the following conclusions may be drawn:

1. The results of the first experiments in 1926 were confirmed.
2. The endurance limit in tension of the welds is not very high, but it is perfectly well defined and therefore can serve as a basis for calculations.

3. For welds of a shape recognized in practice as technically good and carried out with coated electrodes of good quality, there is no need to make a special reduction in the endurance limit on account of possible slight defects.

We did not make fatigue tests in shear, having no structure suitable for that kind of test. It seems to us reasonable to assume that for the metal of our welds, *i.e.*, mild steel, there is some relation between the endurance limits in tension and in shear and that this relation must be the same as for other mild steels.

The recent well-known investigations made under Professor Moore at the University of Illinois have shown that for most steels the endurance limit in shear is on an average 53 per cent of the endurance limit in tension.

CHAPTER III

STUDY OF ELEMENTARY ASSEMBLINGS

Having thus toward the end of the summer of 1926 acquired sufficient knowledge with regard to the metal of the weld itself, we were well prepared to begin the study of elementary assemblies. In structural work the elements may be connected in three ways:

1. *Butt Welded*.—This type of joint, which is not used very frequently nowadays, has been thoroughly studied for boiler work and there is no need to examine it here. We know that a butt-welded joint between mild steel plates easily can be made as strong as the plates themselves.¹

2. *Lap-welded, with or without Gusset-piece*.—This type, which follows the shape of the riveted joint, is the one now most frequently employed, but is gradually being superseded by the third type. Figure 3 shows an example of type 2.

3. *Directly Welded, without Intermediary*.—This type, at present seldom used, must be considered as the welded joint of the future. It is the most economical as regards labor, weight of welding metal, and, above all, weight of structural steel (Fig. 4).

The basic elements of assemblies 2 and 3 are beads or fillets of weld, triangular in section, deposited in the inner angle between the welded pieces.

There is an infinite variety of joints in structural work, but there are only two ways in which the elementary beads or fillets composing them can be loaded or stressed. These we have called:

1. The front weld (see Figs. 5 and 6).
2. The side weld (see Fig. 7).

It might be possible to have an elementary weld in an intermediary condition, but it is quite impossible to imagine one in any other. The problem to be solved therefore is: How does

¹ See the *Annual Reports* of the Association Suisse des Propriétaires d'appareils à vapeur, 1922 and onward.



FIG. 3.

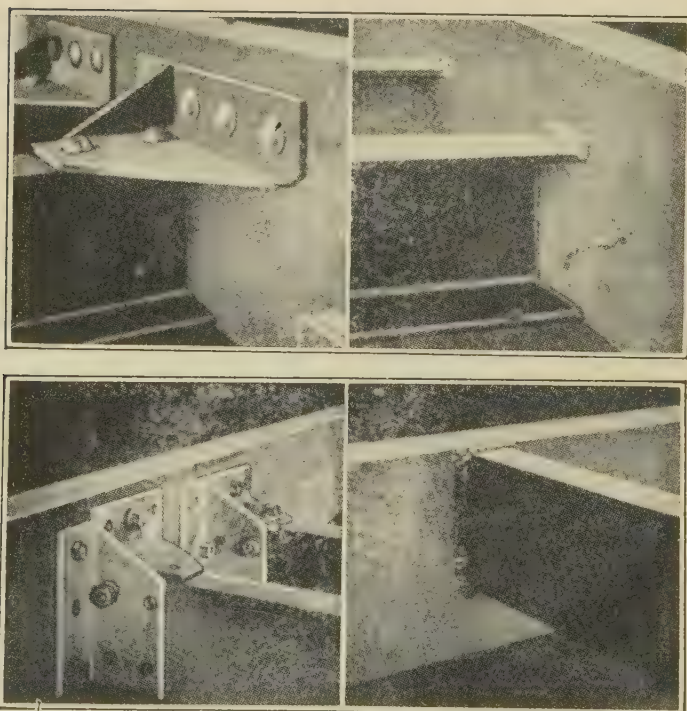


FIG. 4.

the strength of a front weld or a side weld vary according to its shape and dimensions?

Exactly what was known in regard to this in the summer of 1926? From time to time during the preceding four or five years technical papers had dealt with investigations on the strength of front and side welds. Most of these works were based on a few new experiments; several of them referred to two important researches:

1. Humphrys' tests published in *The Iron Age* in 1922.
2. Hoehn's tests published in the *Bulletin de l'Association Suisse des Propriétaires d'appareils à vapeur* in 1922 and 1923. Mr. Hoehn is the director of this association.

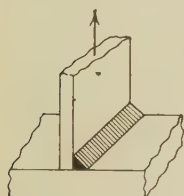


FIG. 5.



FIG. 6.

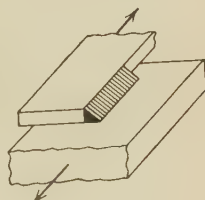


FIG. 7.

Neither Humphrys nor Hoehn had made very numerous tests. Humphrys simply gives the results of his tests, adding very few comments. Hoehn, on the other hand, makes a careful analysis of his results and draws a series of conclusions from them; he lays down laws which, according to him, govern the strength of side and front welds.

Hoehn's conclusions, which have been widely circulated in several languages, are sometimes a little surprising and require verification.¹ Among them we would mention:

1. In front welds the strength per unit distinctly decreases as the thickness of the cover plate increases.
2. In side welds the strength per unit distinctly decreases as the thickness of the cover plate increases.
3. In side welds the strength per unit slightly decreases as the length of the weld increases.²

¹ Mr. Hoehn expresses the strength of his test pieces in tons per square centimeter of the front or side section of the welded elements in direct contact with the weld. This is a simple and convenient method, which we have adopted.

² We will not dwell upon conclusions 3 and 4. Mr. Hoehn seems himself to have abandoned them (Diskussions Bericht nr. 12 ueber elektrisch und autogen geschweisste Konstruktionen, Zurich Congress, 1926).

4. The strength of front and side welds is almost independent of the profile of the bead or fillet of weld.

We attempted to verify these conclusions in two ways:

1. By carefully observing the stressing, straining, and tearing of the welds.

2. By a great number of systematic tests (more than 200) made under very strictly controlled conditions.

1. Stressing, Straining, and Tearing of the Welds.

(a) *Front Welds*.—Let us examine (Fig. 8) a perpendicular section through a front weld of good length. It is obvious that all tensions in the joint are parallel to the plane of the section.

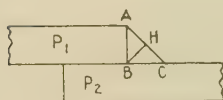


FIG. 8.

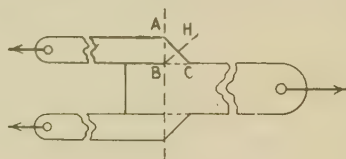


FIG. 9.

They may be investigated by the photo-elasticimetric methods developed by Mesnager in Paris and Coker in London.

We know that the elastic properties of the weld ABC and of the two pieces P_1 and P_2 are practically the same,¹ so that the tensions arising in the plane of Fig. 8 can be studied by means of a transparent xylonite model as shown in Fig. 9. We made our model symmetric in order to avoid abnormal stresses due to bending.

As to Coker's method, which is not widely employed (see *Engineering*, January, 1921), when a beam of ordinary light travels through a piece of glass, celluloid, xylonite, or any other transparent, homogeneous, and isotropic material, it remains unchanged. It has been known for some time that when a piece of transparent material is stressed the beam of light is decomposed into two other beams of plane-polarized light, each of which vibrates in a plane parallel to the direction of one of the two principal normal stresses (that is to say, parallel to the axes of the ellipsoid of elasticity). This phenomenon is not visible to the naked eye.

¹ We found from our experiments in 1925 that the values of Young's modulus for the weld ranged between a maximum of 21,100 and a minimum of 20,300.

If the transparent model (supposed unstressed) is placed between two Nicol prisms whose axes are perpendicular to each other, it will appear opaque, for the second prism will absorb all the light that has passed through the first. When the model is stressed, dark spots are seen, which indicate the points where the principal normal stresses are non-existent or parallel to the axes of the Nicol prisms, and more or less bright spots, colored like Newton rings, which indicate the points where the principal normal stresses are more or less inclined toward the axes of the Nicol prisms.

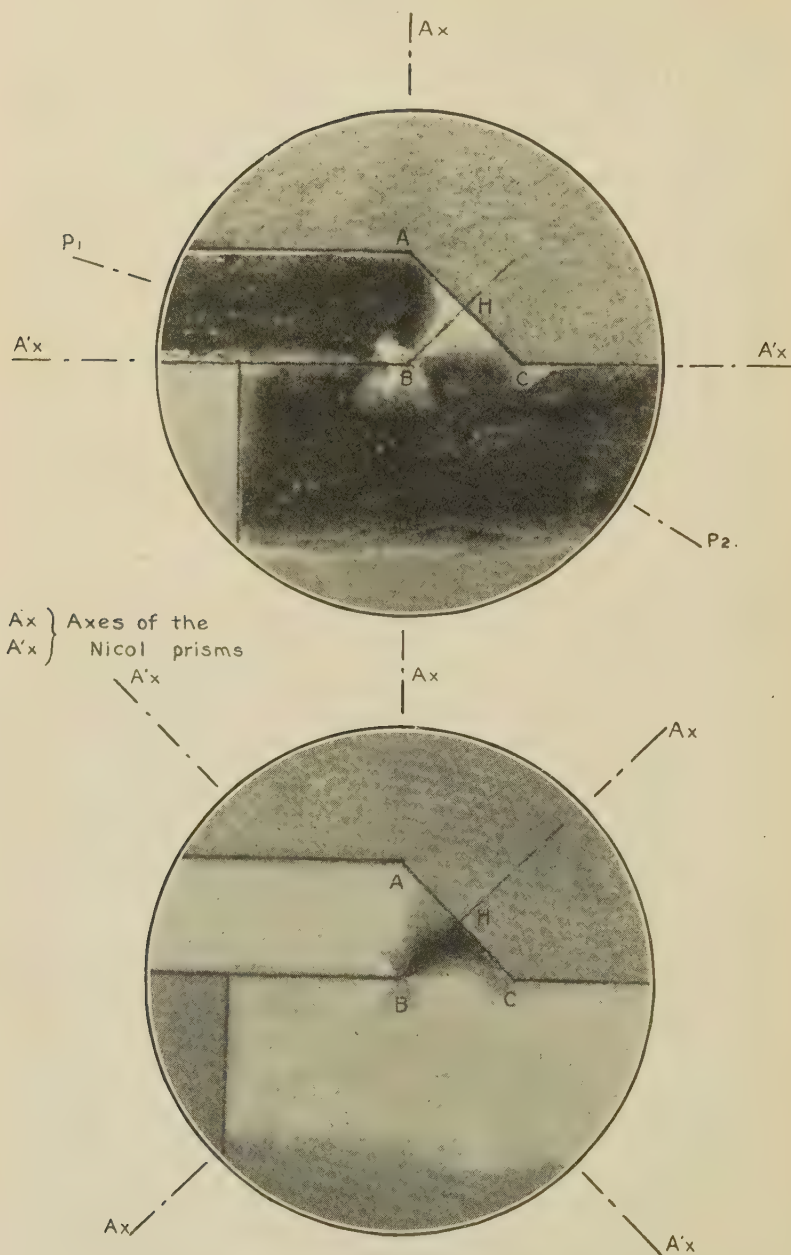
If we now rotate the model, or preferably the two prisms together, we shall see that each part of the model becomes alternately dark or bright according to whether the principal normal stresses in these parts are parallel to the axes of the prisms or make an angle of 45° with them. So the Coker method supplies us with a convenient means of ascertaining exactly the direction of the principal normal stresses at each point in a plane transparent model. Its application in the case of our front weld is very simple.

The illustrations, Figs. 10 and 11, give the appearance of the model of Fig. 9 "under stress." Figure 10 shows the model with its axes parallel to those of the Nicol prisms; inversely, Fig. 11 shows the axes of the Nicol prisms at an angle of 45° .

Let us analyze Fig. 10. Section AB being uniformly dark, we know that this section is loaded in plain tension by forces parallel to P_1 . Section BH being entirely bright, we know that through BH the principal normal stresses make an angle of 45° with the horizontal, *i.e.*, they are perpendicular to BH . BH therefore may be considered to be uniformly loaded in plain tension, like AB . In the base section BC the stresses are rather intricate.

Figure 11 may be analyzed in the same way as Fig. 10 and acts as a check.

A front weld breaks suddenly and without deformation; its shape and position make anything else impossible. We therefore have no means of following either the deformation or the progress of the break. The final shape of the fracture is the only thing we have to go upon. But this fracture is quite typical. The weld breaks along BH . This is not plainly apparent from one isolated specimen as the surface of the fracture is somewhat irregular, but it becomes strikingly evident to anyone who



FIGS. 10 and 11.

examines a number of broken specimens together.¹ So we may say that:

Everything points to the fact that a front weld breaks as if it were uniformly loaded in plain tension through its dangerous section, $B-H$. ✓

From this alone we can deduce the laws governing the strength of front welds. This strength must be proportionate to:

1. The length of the weld.

2. The length of $B-H$, which in turn—

(a) is proportionate to the thickness $A-B$ of the welded piece.

(b) varies according to the proportions of the triangle $A-B-C$;

i.e., according to the relation $\frac{B-C}{A-B} = n$.

(c) varies according to the shape of the surface $B-C$ (convex, flat, or hollow).

We see at once that (2) is incompatible with Mr. Hoehn's conclusions (c) and (d) of 1922–1923. If our own conclusions are correct, it must be possible to calculate exactly the strength of the front weld from its dimensions and the ultimate tensile strength of the metal of the weld. Our experiments gave the following results:

Thickness $A-B$ of the cover plate (P_1) . .	5 mm.	10 mm.	15 mm.
Ultimate tensile strength of the joint noted in square millimeters of the area $B-H = A-B\sqrt{\frac{1}{2}}$	41.5 kg.	39.3 kg.	37.4 kg.
Average of	5 tests	6 tests	5 tests

The ultimate strength of the metal of the weld averaged (according to our numerous tests in 1925–1926) 39.8 kg., the maximum being 42 kg. and the minimum 37 kg. The profile of the weld was an isosceles right-angled triangle; the hypotenuse $B-C$, unmachined, was slightly convex for 5 mm., practically flat for 10 mm., and very slightly hollow for 15 mm. The ends of the weld were milled to give the exact length of 50 mm.

The tests gave practically the same results as the calculations.

(b) *Side Welds*.—For side welds we cannot avail ourselves of photo-elasticimetric methods; most of the acting loads and

¹ In all tests of a theoretical character it is assumed that the section of the weld is an isosceles right-angled triangle. We arranged that this should be approximately so.

stresses do not lie in the plane $A-B-C$, but in planes at right angles to it. Fortunately we can follow the deformation of the weld before the fracture and the progress of the break. The magnitude of the deformation before the break is clearly shown by Fig. 12. Such deformation may be measured by simple devices. The weld breaks as shown in Fig. 13.



FIG. 12.

Let us consider a side weld $A-B-C/A'-B'-C'$, with ends milled to a definite length, and let us pull on the pieces P_1, P_2 . We see first the deformation of the weld shown in Fig. 12; then narrow cracks open simultaneously in C and C' and spread at first slowly toward each other along the curves $C-D$, $C'-D'$, and then suddenly along the straight line $D-D'$, giving the fracture in shear.

The longer the weld the longer the straight line $D-D'$. We see that the specimen broke along a plane $B-B'/D-D'$ ending in two small projecting corners $B-C-D$, $B'-C'-D'$. It is easily seen from a number of broken specimens that the position of the plane $B-B'/D-D'$ is very close to section $B-H$ of the triangle $A-B-C$ in Fig. 8.

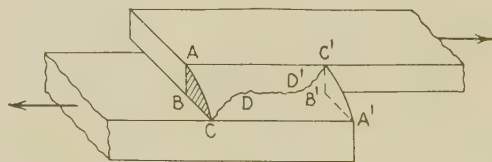


FIG. 13.

If we carried our investigations no further we might say: A side weld breaks just as if it were loaded in plain shear in its dangerous section $B-H$, the shear stress being complicated by secondary local stresses at the ends. This conclusion would lead us to lay down exactly the same laws regarding the strength of side welds as those for front welds.

Actually, things are not so simple. Our numerous tests in 1925 showed that the resistance in shear of the metal of the weld averaged 28.6 kg. per square millimeter. If the weld broke under a plain shearing stress in the area $B-H$, we should be able to calculate exactly its strength, which would be 28.6 kg. $\times$ area $B-H$. We know from our systematic tests that the calculated strength agrees fairly well with the observed strength for welds between thin plates (5 mm. or less), but that it does not agree for larger welds; for plates of 10 mm. the observed strength is about 20 per cent less than the calculated strength.

The strength of the side weld does not increase proportionately to $B-H$ (or to $A-B$, the thickness of the piece). Humphrys' tests, referred to, had already shown this and Hoehn's had confirmed it.

Let us examine the phenomena: For the metal of the weld the elastic limits in shear and in tension (or in compression) differ considerably. Our tests in 1925-1926 gave these limits as $L'' = 15.8$ kg. and $L = 29.8$ kg. per square millimeter. Since the deformation of the joint is pronounced, the elastic limit can be easily noted. After a few preliminary tests we can therefore make specimens so proportioned that the elastic limits of the weld of P_1 and P_2 will be reached for three clearly distinct loads so as to avoid any confusion.

Figure 14 is a diagram of such a test. The ordinates represent the load per square millimeter of section $B-H$ and the abscissa denote the total elongation measured by an Amsler dial extensometer graduated in hundredths of millimeters.

The elastic limit of the weld is marked by the first inflection of the diagram, which is very gradual and difficult to determine very accurately, but which nevertheless is far below 15.8 kg., the elastic limit in plain shear. Three other specimens gave very similar diagrams.

There are then two possible conclusions to be drawn:

(a) The influence of the ends of the weld is considerable.

If this is so, the longer the weld the greater should be its resistance, since the effect of the end must decrease proportionately. Our systematic tests show that the resistance of side welds with milled ends is practically in proportion to the length of the weld, except for short welds in which it is somewhat less. The influence of the ends although very evident is very local and acts only over a very short distance. It is not sufficient to account for the low elastic limit observed.

(b) The shear stresses are very unevenly distributed over the surface $B-H$, so that one point may be stressed to fracture, whereas the mean stress throughout the whole section is only moderate.

But if this is so the break must always start at the same point, the point of maximum tension, and since the fracture develops perfectly gradually it ought to be possible to discover this point. If a specimen is pulled very slowly, the duration of the test being one hour or more, it is observed (see Fig. 13) that the crack $D-D'$ does not appear until the weld is completely broken and that the break has developed from $B-B'$ toward $D-D'$; *i.e.*, from the interior outward.

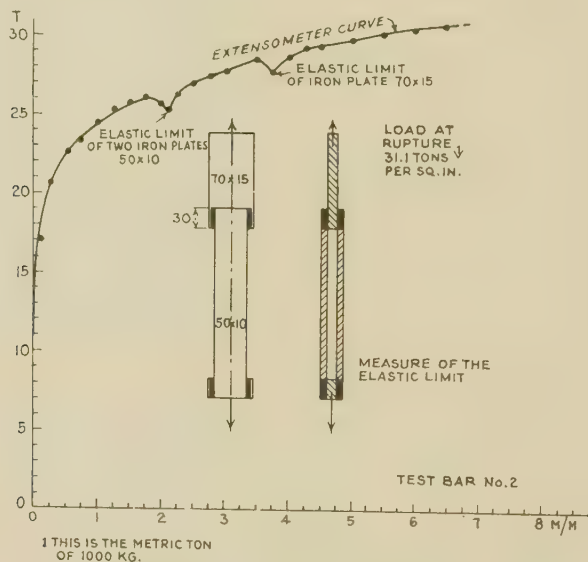


FIG. 14.—Tensile strength test of welded assemblage.

D. Rosenthal, assistant in the Research Laboratory of the University of Brussels, who has been investigating side welds by purely analytical methods, has reached the same conclusion, *viz.*, the tension in shear is greater in $B-B'$ than in $D-D'$. His somewhat complicated formulas give figures that tally approximately with the results of our systematic tests. We found nothing that corroborated Hoehn's conclusion (c).

We may say that as a general rule side welds break in shear under stresses unevenly distributed throughout the plane of their minimum or dangerous section.

Experience shows (see Table 2, Systematic tests) that welds with unmachined ends are much stronger than welds with milled ends. This cannot be attributed only to the absence of the end influence. Unmachined ends act really like two very short front welds which, by themselves, have a very noticeable strength.¹

Before we can formulate the laws governing the strength of side welds we must ascertain:

1. What effect the thickness of the welded pieces has on the strength. (This will be given by our systematic tests.)

2. As for front welds, how is the strength affected by the profile? (To find this we must examine first the influence of

TABLE 2.—SYSTEMATIC TESTS, 1926-1927, ON SPECIMEN SHOWN IN FIG. 8 (Electrode T used. Breaking load given in metric tons of 1,000 kg. per sq. cm. of the section A-B)

Length of each fillet in millimeters	A-B = 5 mm.			A-B = 10 mm.			A-B = 15 mm.		
	B-C =			B-C =			B-C =		
	8-10 mm.	5-6 mm.	†	75 mm.	10 mm.	5 mm.	20 mm.	15 mm.	10 mm.
Side welds (milled ends)									
20	2.12(6)	2.08(3)	0	1.76(3)	1.57(3)	7.02(3)	1.72(3)	1.50(3)	1.29(4)
25	2.12(6)	0	0	0	0	0	0	1.60	0
30	2.20(3)	2.06(3)	0	1.84(3)	1.65(4)	1.07(6)	0	0	1.27(4)
35	2.22(2)	2.10(3)	0	1.825(3)	0	0	1.82(3)	1.65(6)	0
40	2.29(6)	2.10(3)	0	0	1.675(4)	0	0	1.41*(3)	1.25(4)
45		1.96†(3)	0	0	0	0	0	0	0
50	1.95†(3)	1.87(6)†	0	0	1.22†(7)	1.05(1)	0	0	0
Side welds (ends unmachined)									
20	0	2.32(3)	0	0	2.00(3)	0	0	2.28(3)	0
30	2.38	0	0	0	2.09(3)	0	0	1.96(3)	0
Side welds (hollow-milled profile)									
40	0	0	0	0	1.15(3)	0	0	0	0
Front welds (milled ends)									
50	2.93(3)	2.90(3)	0	2.80(3)	2.77(6)	0	0	2.64(5)	0

* Plates fractured.

† Plates yielded, with distortion of the weld.

‡ "Short" welds were frequently unsound in 5 mm.

¹ In practice the ends are always unmachined so that side welds will always be slightly stronger than those of our tests.

the relation $n = B-C/A-B$ in the triangle of the weld, and secondly the shape (convex, flat, or hollow) of the hypotenuse $A-C$ of this triangle.)

Determining the effect of the profile $A-B-C$ on the strength of the weld, if $B-C/A-B = n$ in the triangle of the weld, then

$$B-H = A-B \frac{n}{\sqrt{n^2 + 1}}$$

$B-H$ varies rapidly with n when n is less than 1, but much more slowly when n is greater than 1. Therefore

$n =$	0	$\frac{1}{2}$	$\frac{2}{3}$	1	$1\frac{1}{2}$	2	∞
$B-H/B-C$	0	0.44	0.46	0.70	0.83	0.90	1

We called the welds "isosceles," "short," or "long," according to whether $n = 1$, n is less than 1, n is greater than 1. Since the strength, both of front and side welds, varies with $B-H$, it can be seen that "short" welds should be much weaker than "isosceles," which so far we have been considering, whereas "long" welds should be only slightly stronger. This is completely corroborated by the results of our systematic tests in which we invariably considered simultaneously the three kinds of welds (see Table 2). It follows in practice that:

"Short" welds are weak. A short weld between thin pieces may be dangerous because such welds are difficult to make.

"Long" welds are uneconomical; the slight additional strength that results is out of proportion to the extra outlay in electrode, current, and labor involved.

In practice, therefore, welds should as far as possible be isosceles, at any rate when resisting joints are required.

Let us now consider the hypotenuse $A-C$ of the triangle of the weld. It follows from the foregoing that by making $A-C$ flat, convex, or hollow we get considerable variations in $B-H$, and consequently there should be corresponding variations in the strength of the weld.

According to Hoehn (conclusion (d) of 1922-1923) the shape of the surface is of little importance; in fact, according to him if we pass from a distinctly hollow surface to a very convex one (approximately the quarter of a circle), the strength only increases from 12 to 13 per cent. We tried to reproduce his tests and found that with our specially selected electrode it was very difficult to make a good weld with a distinctly hollow surface. The resistance of such welds was very irregular.

We then milled normal joints between plates 10 mm. thick¹ so as to have a hollow surface one-eighth of a circle. We observed that, compared with unmachined welds in which *A-C* was practically flat, strength was reduced by about one-third, as one might expect (see Table 3, Results of tests).

The difficulty that we found in making hollow welds gave rise to the following idea: that there must be one profile that is easier to make than any other, which therefore we called the "normal" or "natural."

We had observed—and anyone can do the same—that when, as frequently happens in structural work, a welder does repetition work on resisting joints, the profile of his welds approaches more and more to the "normal" or "natural" profile, and in a relatively short time becomes identical with it. Repeated observations showed that this normal profile depends:

- (a) Chiefly on the brand of electrode used.
- (b) Slightly on the thickness of the pieces connected.²
- (c) Hardly at all on the welder.³

With the electrode used in our systematic tests the "normal" profile was flat for plates of medium thickness (10 mm.) and became slightly convex for thinner and very slightly hollow for thicker plates.

Our attention having been directed to this important fact, we noticed that various good coated electrodes used for resisting welds gave a similar profile. On the other hand, certain elec-

¹ Our tests were therefore made under approximately the same conditions as those of Hoehn, who worked with plates 12 mm. thick.

² It may be said to depend upon the diameter of the electrode used and the intensity of current, but we know that in practice if a weld is to be carried out as economically and as quickly as possible, an electrode is chosen with a diameter suitable to the size of the weld, and for this diameter there is a certain current that gives better results than any other. In practice, therefore (and it is the practical point of view that we always consider), the intensity of current and the diameter of electrode are decided by the size of the weld—i.e., by the thickness of the pieces to be welded—and this is the only independent factor.

³ The specimens used in our tests were the work of six or seven different welders who were given no special instructions regarding the profile required. Until we were nearing the end of our tests we thought, like Hoehn, that this was of little importance. On examining our numerous specimens we found that the profile was practically uniform. Table 3, Results of tests, shows that there is very little difference in the resistance of the welds made by the various workmen. This could not be so if there was much difference in the profile.

TABLE 3.—RESULTS OF TESTS
(The ton is the metric ton, of 1,000 kg.)

Date	Operator	Test number	Weld	Plate thick- ness, milli- meters		Length of each fillet in milli- meters	Breaking load		Remarks
				A-B	B-C		Tons	Tons per square centi- meter of A-B	
9-11-26	B.	6	Front weld	5	5-6	50	15.35	3.07	Milled ends
10-29-26	B.	15	Front weld	5	5-6	50	14.20	2.83	Milled ends
10-18-26	Schr.	19	Front weld	5	5-6	50	14.40	2.88	Milled ends
10-23-26	B.	18	Front weld	5	8-10	50	14.72	2.94	Milled ends
10-23-26	B.	17	Front weld	5	8-10	50	14.50	2.90	Milled ends
10-23-26	B.	16	Front weld	5	8-10	50	14.35	2.87	Milled ends
9-23-26	B.	23	Front weld	10	15	50	28.20	2.82	Milled ends
9-23-26	B.	25	Front weld	10	15	50	28.20	2.82	Milled ends
9-23-26	B.	26	Front weld	10	15	50	28.00	2.80	Milled ends
9- 8-26	B.	20	Front weld	10	10	50	27.85	2.78	Milled ends
9- 8-26	B.	22	Front weld	10	10	50	26.40	2.64	Milled ends
9- 8-26	B.	21	Front weld	10	10	50	28.00	2.80	Milled ends
9- 8-26	B.	24	Front weld	10	10	50	27.8	2.78	Milled ends
12- 8-26	G.	27	Front weld	15	15	50	36.70	2.45	Milled ends
12- 8-26	G.	28	Front weld	15	15	50	44.00	2.93	Milled ends
1-17-27	Mtt.	29	Front weld	15	15	50	40.80	2.72	Milled ends
1-17-27	Mtt.	30	Front weld	15	15	50	42.00	2.80	Milled ends
1-17-27	Mtt.	31	Front weld	15	15	50	34.30	2.30	Milled ends
10-13-26	G.	47	Side weld	5	8-10	20	8.20	2.05	Milled ends
10-13-26	G.	48	Side weld	5	8-10	20	8.70	2.17	Milled ends
10-13-26	G.	49	Side weld	5	8-10	20	8.40	2.10	Milled ends
10-28-26	B.	59	Side weld	5	8-10	20	8.20	2.05	Milled ends
10-28-26	B.	60	Side weld	5	8-10	20	8.65	2.16	Milled ends
10-28-26	B.	61	Side weld	5	8-10	20	8.55	2.14	Milled ends
10-13-26	G.	52	Side weld	5	8-10	25	10.25	2.05	Milled ends
10-13-26	G.	51	Side weld	5	8-10	25	10.20	2.04	Milled ends
10-13-26	G.	50	Side weld	5	8-10	25	10.20	2.04	Milled ends
10-28-26	B.	62	Side weld	5	8-10	25	10.98	2.19	Milled ends
10-28-26	B.	63	Side weld	5	8-10	25	10.60	2.12	Milled ends
10-28-26	B.	64	Side weld	5	8-10	25	11.15	2.23	Milled ends
10-13-26	G.	57	Side weld	5	8-10	35	15.50	2.20	Milled ends
10-13-26	G.	58	Side weld	5	8-10	35	15.75	2.25	Milled ends
10-28-26	B.	65	Side weld	5	8-10	30	12.98	2.08	Milled ends
10-28-26	B.	66	Side weld	5	8-10	30	13.60	2.18	Milled ends
10-28-26	B.	67	Side weld	5	8-10	30	14.90	2.38	Milled ends
10-13-26	G.	53	Side weld	5	8-10	40	16.65	2.08	Milled ends
10-13-26	G.	54	Side weld	5	8-10	40	18.98	2.77	Milled ends
10-13-26	G.	55	Side weld	5	8-10	40	16.70	2.09	Milled ends
10-28-26	B.	68	Side weld	5	8-10	40	18.08	2.26	Milled ends
10-28-26	B.	69	Side weld	5	8-10	40	18.25	2.28	Milled ends
10-28-26	B.	70	Side weld	5	8-10	40	18.45	2.31	Milled ends
9- 6-26	B.	32	Side weld	5	8-10	50	19.00	1.90†	Milled ends
9- 6-26	B.	38	Side weld	5	8-10	50	19.50	1.95†	Milled ends
9- 6-26	B.	41	Side weld	5	8-10	50	19.60	1.96†	Milled ends
2-28-27	Mtt.	144	Side weld	5	8-10	30	14.03	2.34	Ends unmachined
2-28-27	Mtt.	145	Side weld	5	8-10	30	14.92	2.48	Ends unmachined
2-28-27	Mtt.	146	Side weld	5	8-10	30	13.92	2.32	Ends unmachined
2-28-27	B.	132	Side weld	5	8-10	20	8.38	2.09	Milled ends
2-28-27	B.	133	Side weld	5	5-6	20	8.05	2.01	Milled ends
2-28-27	B.	134	Side weld	5	5-6	20	8.56	2.14	Milled ends
2-28-27	B.	135	Side weld	5	5-6	30	11.90	1.98	Milled ends
2-28-27	B.	136	Side weld	5	5-6	30	12.72	2.12	Milled ends
2-28-27	B.	137	Side weld	5	5-6	30	12.85	2.14	Milled ends
1-17-27	Mtt.	44	Side weld	5	5-6	35	15.10	2.16	Milled ends
1-17-27	Mtt.	45	Side weld	5	5-6	35	14.00	2.00	Milled ends
1-17-27	Mtt.	46	Side weld	5	5-6	35	15.05	2.15	Milled ends
2-28-27	B.	138	Side weld	5	5-6	40	17.00	2.12	Milled ends
2-28-27	B.	139	Side weld	5	5-6	40	17.00	2.12	Milled ends
2-28-27	B.	140	Side weld	5	5-6	40	17.40	2.18	Milled ends

† Plates yielded, with distortion of the weld.

TABLE 3.—RESULTS OF TESTS.—(Continued)

Date	Operator	Test number	Weld	Plate thickness, millimeters		Length of each fillet in millimeters	Breaking load		Remarks
				A-B	B-C		Tons	Tons per square centimeter of A-B	
3-11-27	Mldrs.	192	Side weld	5	5-6	45	16.63	1.85†	Milled ends
3-11-27	Mldrs.	193	Side weld	5	5-6	45	18.06	2.00†	Milled ends
3-11-27	Mldrs.	194	Side weld	5	5-6	45	18.16	2.02†	Milled ends
9-6-26	B.	33	Side weld	5	5-6	50	18.50	1.85†	Milled ends
9-6-26	B.	34	Side weld	5	5-6	50	18.30	1.84†	Milled ends
9-6-26	B.	35	Side weld	5	5-6	50	18.20	1.82†	Milled ends
9-6-26	B.	36	Side weld	5	5-6	50	18.50	1.85†	Milled ends
9-6-26	B.	37	Side weld	5	5-6	50	18.50	1.85†	Milled ends
9-6-26	B.	39	Side weld	5	5-6	50	18.70	1.87†	Milled ends
9-6-26	B.	41	Side weld	5	5-6	50	19.60	1.96†	Milled ends
9-6-26	B.	42	Side weld	5	5-6	50	18.00	1.80†	Milled ends
9-6-26	B.	43	Side weld	5	5-6	50	18.80	1.88†	Milled ends
8-28-27	Mldrs.	165	Side weld	5		20	9.82	2.42	Ends unmachined
8-28-27	Mldrs.	166	Side weld	5		20	9.60	2.40	Ends unmachined
8-28-27	Mldrs.	167	Side weld	5		20	8.80	2.20	Ends unmachined
12-9-26	B.	95	Side weld	10	15	20	14.70	1.83	Milled ends
12-9-26	B.	96	Side weld	10	15	20	14.20	1.77	Milled ends
12-9-26	B.	97	Side weld	10	15	20	13.20	1.65	Milled ends
12-9-26	B.	98	Side weld	10	15	30	23.00	1.92	Milled ends
12-9-26	B.	99	Side weld	10	15	30	22.00	1.83	Milled ends
12-9-26	B.	100	Side weld	10	15	30	21.80	1.81	Milled ends
12-9-26	B.	101	Side weld	10	15	35	24.90	1.77	Milled ends
12-9-26	B.	102	Side weld	10	15	35	26.20	1.88	Milled ends
12-9-26	B.	103	Side weld	10	15	35	25.70	1.83	Milled ends
12-3-26	B.	83	Side weld	10	10	20	12.50	1.53	Milled ends
12-3-26	B.	84	Side weld	10	10	20	12.45	1.56	Milled ends
12-3-26	B.	85	Side weld	10	10	20	13.20	1.65	Milled ends
12-3-26	B.	87	Side weld	10	10	30	21.00	1.75	Milled ends
12-3-26	B.	88	Side weld	10	10	30	19.10	1.59	Milled ends
12-3-26	B.	88	Side weld	10	10	30	19.50	1.62	Milled ends
12-3-26	B.	90	Side weld	10	10	30	19.50	1.62	Milled ends
12-3-26	B.	91	Side weld	10	10	40	27.50	1.72	Milled ends
12-3-26	B.	92	Side weld	10	10	40	27.30	1.71	Milled ends
12-3-26	B.	93	Side weld	10	10	40	26.40	1.69	Milled ends
12-3-26	B.	94	Side weld	10	10	40	26.00	1.62	Milled ends
9-1-26	B.	71	Side weld	10	10	50	24.65	1.25†	Milled ends
9-1-26	B.	72	Side weld	10	10	50	24.60	1.23†	Milled ends
9-1-26	B.	73	Side weld	10	10	50	23.90	1.19†	Milled ends
9-1-26	B.	74	Side weld	10	10	50	24.85	1.24†	Milled ends
9-1-26	B.	75	Side weld	10	10	50	25.10	1.25†	Milled ends
9-1-26	B.	76	Side weld	10	10	50	23.90	1.19†	Milled ends
9-1-26	B.	81	Side weld	10	10	50	24.30	1.21†	Milled ends
5-3-27	Mtt.	168	Side weld	10	10	30	24.60	2.05	Ends unmachined
5-3-27	Mtt.	169	Side weld	10	10	30	25.50	2.12	Ends unmachined
5-3-27	Mtt.	170	Side weld	10	10	30	24.95	2.08	Ends unmachined
1-28-27	Mldrs.	162	Side weld	10	10	20	16.30	2.02	Ends unmachined
1-28-27	Mldrs.	163	Side weld	10	10	20	16.10	2.01	Ends unmachined
1-28-27	Mldrs.	164	Side weld	10	10	20	15.80	1.98	Ends unmachined
3-9-27	Mldrs.	174	Side weld	10	10	40	20.60	1.29	Hollow-milled profile
3-9-27	Mldrs.	175	Side weld	10	10	40	17.00	1.06	Hollow-milled profile
3-9-27	Mldrs.	176	Side weld	10	10	40	17.80	1.12	Hollow-milled profile
3-2-27	Mldrs.	177	Side weld	10	5	20	8.50	1.06	Milled ends
3-2-27	Mldrs.	178	Side weld	10	5	20	7.35	0.92	Milled ends
3-2-27	Mldrs.	179	Side weld	10	5	20	8.60	1.08	Milled ends
2-11-27	Mldrs.	129	Side weld	10	5	30	12.50	1.04	Milled ends
2-11-27	Mldrs.	130	Side weld	10	5	30	11.50	0.88	Milled ends
2-11-27	Mldrs.	131	Side weld	10	5	30	12.63	1.06	Milled ends
3-2-27	Mldrs.	180	Side weld	10	5	30	15.75	1.31	Milled ends
3-2-27	Mldrs.	181	Side weld	10	5	30	12.75	1.01	Milled ends
3-2-27	Mldrs.	182	Side weld	10	5	30	12.98	1.08	Milled ends
9-1-26	B.	82	Side weld	10	5	50	21.10	1.05	Milled ends

† Plates yielded, with distortion of the weld.

TABLE 3.—RESULTS OF TESTS.—(Continued)

Date	Operator	Test number	Weld	Plate thickness, millimeters		Length of each fillet in millimeters	Breaking load		Remarks
				A-B	B-C		Tons	Tons per square centimeter of A-B	
2-23-27	Moldrs.	153	Side weld	15	20	20	21.20	1.77	Milled ends
2-23-27	Moldrs.	154	Side weld	15	20	20	20.65	1.72	Milled ends
2-23-27	Moldrs.	155	Side weld	15	20	20	20.30	1.69	Milled ends
2-23-26	Moldrs.	156	Side weld	15	20	35	38.00	1.82	Milled ends
2-23-26	Moldrs.	157	Side weld	15	20	35	37.80	1.80	Milled ends
2-23-26	Moldrs.	158	Side weld	15	20	35	38.86	1.85	Milled ends
1-14-27	Mtt.	104	Side weld	15	15	20	17.95	1.50	Milled ends
1-14-27	Mtt.	105	Side weld	15	15	20	17.85	1.49	Milled ends
1-14-27	Mtt.	106	Side weld	15	15	20	18.00	1.50	Milled ends
1-14-27	Billy	107	Side weld	15	15	25	24.00	1.60	Milled ends
1-14-27	Billy	108	Side weld	15	15	25	24.70	1.63	Milled ends
1-14-27	Billy	109	Side weld	15	15	25	25.40	1.69	Milled ends
1-21-27	Mtt.	111	Side weld	15	15	35	33.90	1.57	Milled ends
1-21-27	Mtt.	112	Side weld	15	15	35	32.75	1.55	Milled ends
1-21-27	Mtt.	113	Side weld	15	15	35	31.90	1.52	Milled ends
1-29-27	Moldrs.	114	Side weld	15	15	35	36.50	1.73	Milled ends
1-29-27	Moldrs.	115	Side weld	15	15	34	34.50	1.70	Milled ends
1-29-27	Moldrs.	116	Side weld	15	15	32.2	32.70	1.70	Milled ends
3-9-27	Moldrs.	198	Side weld	15	15	45	42.50	1.57*	Milled ends
3-9-27	Moldrs.	199	Side weld	15	15	45	35.90	1.32*	Milled ends
3-9-27	Moldrs.	200	Side weld	15	15	45	41.50	1.53*	Milled ends
1-13-27	Mtt.	159	Side weld	15	15	20	25.80	2.15	Ends unmachined
1-13-27	Mtt.	160	Side weld	15	15	20	23.90	2.40	Ends unmachined
1-13-27	Mtt.	161	Side weld	15	15	20	27.20	2.27	Ends unmachined
2-23-27	Mtt.	147	Side weld	15	15	30	35.15	1.95	Ends unmachined
2-23-27	Mtt.	148	Side weld	15	15	30	34.90	1.94	Ends unmachined
2-23-27	Mtt.	149	Side weld	15	15	30	35.9	1.99	Ends unmachined
12-7-26	B.	117	Side weld	15	15	20	15.5	1.29	Milled ends
12-7-26	B.	118	Side weld	15	10	20	15.35	1.28	Milled ends
12-7-26	B.	119	Side weld	15	10	20	15.25	1.27	Milled ends
12-7-26	B.	120	Side weld	15	10	20	15.5	1.29	Milled ends
12-7-26	B.	121	Side weld	15	10	30	22.4	1.25	Milled ends
12-7-26	B.	122	Side weld	15	10	30	22.4	1.25	Milled ends
12-7-26	B.	123	Side weld	15	10	30	24.5	1.36	Milled ends
12-7-26	B.	124	Side weld	15	10	30	22	1.22	Milled ends
12-7-26	B.	125	Side weld	15	10	40	28.9	1.21	Milled ends
12-7-26	B.	126	Side weld	15	10	40	30.7	1.28	Milled ends
12-7-26	B.	127	Side weld	15	10	40	29.5	1.23	Milled ends
12-7-26	B.	128	Side weld	15	10	40	31.5	1.31	Milled ends

* Plates fractured.

trodes made especially for welding thin plates or for other light work gave a very hollow profile. The weld was very neat and clean, but rather weak. We have seen that the best type for resisting welds, both as regards safety and economy, is the "isosceles."

Repetition work is being more and more organized on the bonus system; the workman is tempted to work more quickly, using less electrode and current, but he is faced with the possibility of being fined if his welds do not pass the control tests. So he steers a medium course. Our experiments have shown that the happy medium is the isosceles type.

It seems, therefore, that for actual welded structures, in which the considerations of safety and economy enter largely, the profile of the weld will vary only slightly and will never be far from the normal isosceles. We found this to be so in several large structures.¹

Behavior of the Welds under Dynamic Stresses.—Front welds break with very little deformation, so for these welds the amount of work consumed must necessarily be small. In side welds there is much deformation before the break, so that one would expect the amount of work required to cause the fracture to be considerable.

Our tests in 1925–1926 showed the amount of work consumed in breaking our specimens when subjected to a slow pull or to a sudden shock. We did not know what it was in shear, the chief stress to which side welds are subjected.

We therefore carried out tests with specimens proportioned as shown in Fig. 14 containing four welds, 5×5 mm. normal isosceles profile, milled to a length of 20 mm., connecting plates of mild steel. The electrode used was the same as in our systematic tests. Slow pulling gave the following results:

Number of specimen	Work consumed, ¹ kilogram-meters	Ultimate tensile strength, tons ²
204	42	7.70
205	36.3	7.75
206	31.5	7.80
Average.....	36.0	7.75

¹ Calculated by computing the stress-strain diagram.

² This is the metric ton, of 1,000 kg.

¹ The tests made by the Metropolitan Gas Company in Melbourne before constructing their large welded gas holders led them to use this profile. They worked like ourselves with coated electrodes of good quality (*Journal of the American Welding Society*, January, 1927).

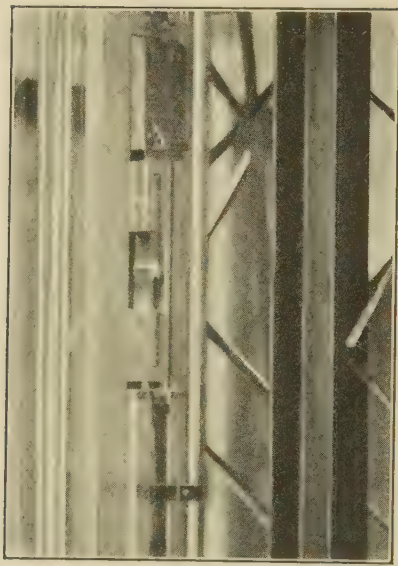


FIG. 15.

We then tried exactly similar tests on our Amsler Universal ram impact machine adapted for the purpose, as shown in Fig. 15, with the following results:

Number of specimen	Work consumed, kilogram-meters
201	54.5
202	45.0
203	49.5
Average.....	50.0

The dynamic strength in shear increases very rapidly as the length of the weld increases. If the length of the weld is increased to 35 mm., we find:

Number of specimen	Work consumed, kilogram-meters
229	107 } Fracture in the plates; weld unbroken. 99 } 158 Special steel plates; weld broken.
230	
231	

Let us substitute for our welds in specimens 201 to 206 a good mild steel rivet 12 mm. in diameter. To prevent tearing, the plates must be of 60-kg.m. steel. This gives us, with the 12-mm. rivet, double sheared, pulled slowly:

Number of specimen	Ultimate tensile strength, tons ¹	Work consumed, kilogram-meters
205a	8.7	35.5
206a	7.8	31.5
Average.....	8.15	33.5

¹ This is the metric ton, of 1,000 kg.

With an impact test, we found:

Number of specimen.....	206	207	208	209	210	211
Work consumed, kilogram-meters....	51	48.5	51	51	51	48.5
Average.....	50 kg.m.					

The same assembling, with mild steel plates, consumed approximately 100 kg.m. in breaking. In this case the result represents the amount of work required both in breaking the rivet and in partially tearing the plates. It is very similar to that given by the welded specimens Nos. 229 and 230. From these figures two conclusions are apparent:

(a) The smallness of the side weld capable of satisfactorily replacing a good rivet.

(b) The high dynamic strength of a side weld.

CHAPTER IV

SYSTEMATIC TESTS

Choice of Electrode.—Our tests in 1925-1926 showed how much depended on this choice. In order to have results that could be compared one with another the same electrode must be used for all the tests. In order that the results of our tests might be applicable in shop practice the electrode used must be a typical average good quality electrode such as is used in structural work—so far as possible an electrode likely to be considered good for some time to come. The electrode chosen was T. Our tests of 1925-1926 showed that this electrode deposited a mild steel with the following mechanical characteristics:

- Z = ultimate tensile strength.
 - (a) measured on specimens entirely built up with the electrode
6 specimens; average, 39.8 kg. per sq. mm. (maximum 42 kg., minimum 37 kg.).
 - (b) measured on welded bits of 10 to 30 mm. diameter
35 specimens; average, 38.4 kg. (maximum 40.5 kg., minimum 35.5 kg.), of which 18 gave between 39 and 42 kg.
- L = apparent elastic limit (yield point)
6 specimens; average, 29.8 kg. per square millimeter (maximum 32.7 kg., minimum 27.6 kg.).
- E = elastic modulus in tension
2 specimens; average, 20.550 kg. per square millimeter (maximum 20.800 kg., minimum 20.300 kg.).
- Z'' = ultimate strength in shear
8 bars; average, 28.66 kg. per square millimeter (maximum 29.2 kg., minimum 28 kg.).
- L'' = elastic limit in shear
2 torsion tests; both gave 15.8 kg. per square millimeter.
- E'' = elastic modulus in shear
Same specimens; both gave 15.4 kg. per square millimeter.
- A^1 = elongation
4 specimens, 10×100 mm.; average, 19.4 kg. per square millimeter (maximum 25 kg., minimum 14.2 kg.).

These figures sufficiently bear out our conclusions of 1925-1926. It will be seen that the mechanical properties of the metal deposited by the electrode T are very similar to those of

most structural steels. In other words, steel structures are welded with the same metal as the component parts.

The question might be asked, "Would it not be more advantageous or more economical to use electrodes giving a greater strength; *e.g.*, Sp.B., which is 25 per cent stronger than T?" If we consider the conditions most frequently occurring in actual work, we can confidently reply, "No." Little would be gained, and the weld might be unsafe. As a matter of fact, a front weld might be made about one-fifth shorter or the load increased in the same proportion provided that the welded pieces are loaded longitudinally; *i.e.*, parallel to their grain.

In making our tests we noticed that the strength of structural steel across the grain was very irregular and often insufficient to stand the strain set up by a weld such as Sp.B.

Side welds cannot be made much shorter because of the poor strength of structural steels in longitudinal shear. Before breaking, a weld such as T strains the adjacent metal of the structure almost to breaking. If we make a weld of the same strength with Sp.B. (it will be shorter than the weld with T), the specimen breaks in shear in the welded piece alongside the weld. So in practice it is impossible to make full use of the strength of such an electrode.

To sum up, from the point of view of safety and economy it is inexpe-

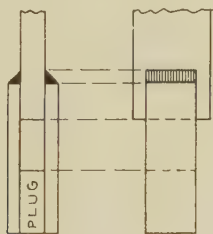
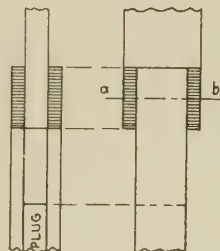


FIG. 16.



SECTION a-b



FIG. 17.

dient to weld low-grade metals like structural steel with high quality electrodes.

Shape of the Specimens.—Our test pieces were built up as shown in Figs. 16 and 17. We chose this symmetrical shape—it had been used by others before us—in order to avoid secondary bending stresses.

The welds were carried out by several different workmen, at least six. It will be seen from Table 3, Results of tests, that there is very little difference in the ultimate strength of the welds made by these men. This shows that in repetition work, such as the making of our specimens, the personal element tends to disappear.

Table of Results.—The results of our test are given in two tables. Table 2 gives the average result of each series of tests in order to show the laws governing the strength of the welds. The figures in the boxes give the average ultimate strength, and the number of tests on which this average was taken is shown by the figures in parentheses following.

Table 3 gives the result of each of the tests on which the averages shown in Table 2 were obtained. It shows how near the individual specimens in each series were to the average and in this way demonstrates the regularity of the results obtained.

It should be noted that specimens in the same series tested on different days were as a rule made by different workmen. A study of the figures in these two tables confirms the conclusions heretofore reached.

Conclusions.—From the foregoing, practical rules (which we put forward tentatively) for calculating structural joints can be deduced. They are very simple, and may be stated as follows:

1. With the exception of butt-welded joints, all structural joints can be made with elementary beads or fillets of weld which may be in one or other of the three positions which we have called front welds, side welds, and intermediary welds.

2. These fillets or beads are made with electrodes which, in addition to the usual properties required in a good electrode, possess the following characteristics:

- (a) The metal deposited is mild steel with a strength of 38 to 40 kg.m. per square millimeter and 15 to 20 per cent elongation.

- (b) The "normal" or "natural" profile of the deposited bead is flat or slightly convex, never appreciably hollow.

3. This being so, the strength of front welds may be safely taken as 2.6 tons per square centimeter of the section of the piece in direct contact with the weld. For ordinary structural steel with an average tensile strength of 40 kg. per square millimeter we may say that a front weld is equal to 65 per cent, or two-thirds of the section of the piece in direct contact with the weld.

4. This being so, the strength of side welds may be safely taken as between 1.6 and 2 tons per square centimeter of the section of the piece in direct contact with the weld, according to the thickness of the cover plate. For ordinary structural steel with an average tensile strength of 40 kg. per square millimeter, we may say that a side weld is between 50 per cent, or one-half, and 40 per cent, or two-fifths, of the section of the piece in direct contact with the weld, according to the thickness of the cover plate.

5. It is advisable to treat "intermediary" welds as side welds if they are inclined at an angle of 60° or less with the axis of the welded pieces.

6. If the structure, as is most frequent, is to be subjected only to static loads, front welds will be primarily employed, because of their great strength and economy. On the contrary, when dynamic loads have to be taken into consideration, side welds are preferable.

CHAPTER V

APPLICATION TO STRUCTURAL WORK

Very simple rules having been laid down for the calculation of joints, we had still to prove their accuracy by applying them to structures. This was done in the autumn of 1927.

Preliminary Question.—When two structural steel elements of the same section are assembled by means of rivets on a gusset plate or a cover plate, the strength of the resulting whole is not equal to that of the metal multiplied by the section of each element; it is distinctly less. There are two reasons for this: the rivet loss; the pieces are loaded more or less eccentrically.

In welded joints there are no rivet holes, but the eccentric loading remains to the full extent in assemblies of (b) type and to a smaller extent in those of (c) type.

The loss of strength because of the rivets can be calculated easily, but not that due to eccentric loading. In order to ascertain this we must make tests, and these tests are necessary because, on the one hand, this loss has not so far as we know been clearly ascertained for riveted joints and, on the other hand, it is unnecessary and uneconomical to make welds stronger than the welded pieces themselves. We therefore carried out experiments with L-shaped, T-shaped, and U-shaped

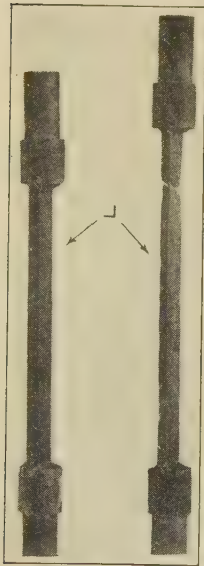


FIG. 18.

pieces. First we fixed them by a front weld to a mild steel cylinder (Fig. 18) so that the neutral axis of the piece fell exactly in line with the axis of the cylinder. By putting a fillet of weld all around the piece it is easy to make a joint much stronger than the piece itself.

If a test specimen that has been built up in this way is pulled, the metal of the piece is in the best condition for developing its

full strength, and we thus ascertained an ultimate tensile strength Z_1 per square millimeter. To check this we also pulled flat test pieces cut out of the complete specimen and obtained a tensile strength Z_1 , which generally was very close to Z .

Then our shapes were assembled in pairs on a thick plate, forming a gusset piece, by side and front welds stronger than the shapes themselves (Fig. 19). These assemblings were made symmetrical in order to avoid abnormal bending stresses. This time we observed a tensile strength Z_2 per square millimeter, which was naturally inferior to Z_1 or Z'_1 . In view of the normal irregularity of structural steel we were careful that all specimens of the same section should be taken from the same bar. The results of these tests are summarized in Table 4. The following conclusions result:

TABLE 4.—MAXIMUM AVAILABLE STRENGTH OF L, T, AND U SHAPES, IN KILOGRAMS PER SQUARE MILLIMETER

Shape		Twin pieces welded on a gusset	Butt welded on a cylinder, axes in line	Small cut-out bands
L	1	38.00	40.3	
30 × 30 × 4	2	36.40	40.6	
L	1	39.00	40.9	
50 × 50 × 5.1	2	35.20 ¹	40.9	
L	1	40.00		42.2
70 × 70 × 7.5	2	39.25		42.6
T	1	38.10	38.4	40.0
53 × 28 × 6.6	2	37.60	37.8	37.9
T	1	37.20	37.8	38.1
50 × 50 × 6.6	2	37.40	37.8	38.3
U	1	37.30		37.4
80 × 44 × normal	2	37.30		36.8

¹ Weakness and distortion of the L.

The reduction in strength due to eccentric loading varies according to the profile, but it is always very small.

Nevertheless the distribution of the stresses through the section, at least when approaching the ultimate, seems to be very uneven. Figures 19 and 20 show to what extent the pieces bend toward each other before breaking and how much more the fibers in contact with the gusset piece are strained than those opposite.

We note that the reduction in strength is never more than 10 per cent and that it becomes negligible in profiles, such as the normal U-shape or the short-armed T-shape. According to the factor of safety used in structural work, this reduction in strength can be neglected.

We have therefore to calculate our welds for the full strength of the pieces; *i.e.*, to give an ultimate strength equal to the section of the piece in square millimeters multiplied by 40 kg.m.

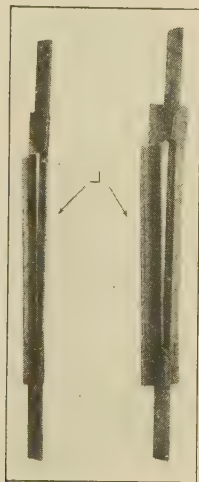


FIG. 19.

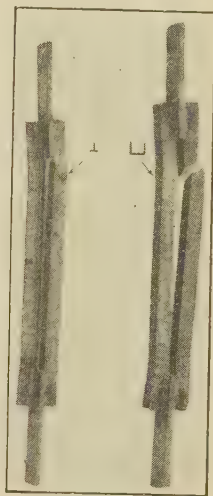


FIG. 20.

Systematic Tests on Structural Steels.—We carried out a series of welds calculated as has been stated in the foregoing and so proportioned that they were 15 per cent weaker than the welded pieces, so that the fracture was sure to occur in the weld. A front weld alone will never give the required strength; in addition there must always be side welds of a certain length. Side welds alone may give the required strength provided they are sufficiently long. We made tests in both these ways.

In non-symmetrical shapes we arranged that the center of resistance of the welds should be close to the neutral axis; of the shape, for instance, in angle pieces, one of the side welds was about three times as strong as the other.

In shop practice no marks are made to show the exact size of the beads or fillets of weld. The number of the electrode is

chosen suitable for the work to be done, and one, two, or three layers of metal are deposited.

In order to work under the same conditions as those in practice, with an electrode No. 8 we deposited fillets or beads of weld having the normal isosceles profile and of a size corresponding to one, two, or three successive layers. Such fillets or beads are very similar to the 5-, 10-, 15-mm. beads that we studied in our systematic tests on elementary assemblings. The results of all the tests carried out in this way are summarized in Table 5.

TABLE 5.—GUSSET JOINTS OF L, T, AND U SHAPES, SIDE WELDS ONLY

Shapes	Electrode T (No. 8)		Strength in tons	
	Length of the fillets, millimeters	Number of layers	Calculated	Observed
L	1	30	75.2	15.3
30 × 30 × 4	2			15.3
L	1	60	38.4	38.2
50 × 50 × 5.5	2			38.3
L	1	120	80.0	80.5
70 × 70 × 7.5	2			80.3
T	1	90	36.0	30.3
53 × 28 × 6	2			¹
T	1	130	50.4	45.0
50 × 50 × 6	2			45.0
U	1	120	88.0	86.3
80 × 44 × normal	2			86.5
	3			87.5

¹ Weakness in one of the T's.

It is remarkable how close the observed strength is to the calculated strength when side welds alone are used. There is more discrepancy between them when both side and front welds are used. The observed strength is then slightly below the calculated strength. In the case of T-shaped pieces the difference is appreciable.

The reason for this is that in side welds the deformation before fracture is much greater than in front welds. When the assembling first begins to stretch, the two welds are no longer loaded in the calculated proportions; the front weld, because of its great stiffness, works harder and harder with the increasing

load. So as a very general rule we find the fracture originating in the front weld, although it is by far the strongest part of the whole. This uneven loading, which is especially obvious when the specimen is strained to the ultimate, already exists in the elastic period.

What allowance must be made for this, and which part must be strengthened and to what extent? For L- and U-shaped

TABLE 6.—GUSSET JOINTS OF L, T, AND U SHAPES, FRONT AND SIDE WELDS COMBINED

		Electrode T (No. 8)			Strength in tons	
Shapes		Front weld, number of layers	Side weld		Calcu- lated	Ob- served
			Length of the fillets	Number of layers		
L	1	1	20	2 + 1	17.0	17.6 ₁
30 × 30 × 4	2					
L	1	1	30	3 + 1	33.8	31.4
	2					30.7
	3					31.7
50 × 50 × 5.5	4					31.9
L	1	1	60	3 + 1	65.4	63.5
70 × 70 × 7.5	2					63.0
T	1	1	35	1 + 1	30.4	24.8
	2					24.5
	3					26.1
52 × 28 × 6	4					25.6
T	1	1	50	1 + 1	33.0	26.0
50 × 50 × 5	2					25.0
U	1	1	65	2 + 2	70.0	67.5
48 × 5	2					69.0
80 × 5						
T	1	Reinforced, detail 1	55	1 + 1	41.5	39.5
50 × 50 × 5.5	2	Reinforced, detail 2	50	1 + 1	30.0	28.6

¹ Fracture in the L.

pieces where the difference between the calculated strength and the observed strength is small (less than 10 per cent), obviously the easiest thing will be to lengthen the side welds. For T-shaped pieces, where the difference may be as great as 20 per cent, the side welds can be lengthened to make up for this, but the assembling will be noticeably increased in consequence. Frequently the assembling is of such a shape that the front weld can be strengthened, as was done in our tests of Sept. 30, 1927 (Table 6). In this way a more compact assembling is obtained.

We now know all the elements necessary to calculate accurately and with safety lap-welded or gusset-welded assemblings of shapes under static tension loads; *i.e.*, in cases where the full strength of the metal can be utilized. Having carefully studied this first case we can deal much more quickly with the other cases of normal occurrence in structural work.

Compression Loading.—If the pieces are short they can be dealt with as if pulled. If, on the other hand, as is usually the case, they are slender, the column effect will have to be taken into consideration.

Will the collapsing load of a long compressed member be greater or less for a welded structure than for a riveted one? This question must be raised in the interests of economy, for it is useless to calculate and proportion joints for loads greater than those admissible for the welded parts. The calculation of long pieces under compression in riveted structures has given rise to much discussion. As a rule they are calculated as if with pivoted ends. In some countries and under some administrations it is permissible to calculate these as if they were only 80 per cent as long as they actually are.

The most recent European experiments in this regard were made in 1922 by the "Technische Kommission des Verbandes Schweiz Brücken und Eisenhochbaufabriken." This commission of eminent engineers made a careful study of fourteen structures of different types which were regarded as technically faultless (ten railway bridges, three highway bridges, and one experimental girder). This commission came to the positive conclusion that in most cases long compressed members were strained just as if their ends were actually pivoted. There is no justification for calculating these pieces as being only 80 per cent of their real length.

In 1926 Professor Ros, of Zurich, checked these conclusions by the most modern methods of calculation, particularly those of Engesser and von Karman, and completely confirmed them.

Must the same strictness be observed in the calculation of welded structures? Lap-welded or gusset joints are more rigid than similar riveted joints because in them the inevitable play of the rivets in their holes is non-existent. On the other hand, since these assemblings are smaller and lighter it is to be expected that the elastic deformations will be greater. Experiments alone can throw light on this question.

Therefore, in order to ascertain to what extent the ends of a compressed welded piece may be considered as fixed rather than pivoted, we embarked on a series of tests which led us very quickly to a conclusion. First of all we had to make sure that we had a machine in the laboratory capable of giving perfect fixity of the ends. We chose our horizontal Amsler 30-ton machine for its massive frame and the perfect guidance of its heads.

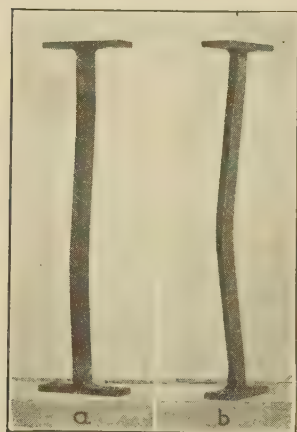


FIG. 21.

From each of the shapes already studied, we cut out pieces exactly 1 m. long and machined the ends square. These pieces were then butt welded onto solid steel plates 25 by 150 by 200 mm., carefully ground so as to lie exactly flat on the head blocks of the machine to which they were bolted (Fig. 21).

We made eight specimens in this way and compressed them till they collapsed. Four of them (one angle, one T, and two U's) withstood a fairly heavy load and gave the S-shape usual in the case of well-fixed ends (Fig. 21-a). The other four (one angle, three T's) collapsed under a moderate load and became C-shaped as if the ends were insufficiently fixed (Fig. 21-b).

We then calculated what was the degree of relative fixity of the ends for each of these specimens. They were too short to be calculated with Euler's formula; for such ratio of slenderness we know that the strength calculated with Tetmayer's formula is very close to the observed strength.

Let us introduce in the formula the observed collapsing load and suppose the length of the piece to be unknown and the ends to be pivoted. If the ends are really pivoted, the length given by the formula ought to be the real length of the piece; *i.e.*, for our specimen 1 m. If the ends are perfectly fixed, the calculated length should be one-fourth of the real length; *i.e.*, for our specimen 0.25 m.

For the four specimens of the first group, we get 0.25 m. $\pm$ 1 c.m., which agrees very well with our observations. For the specimens of the second group, we get values lying between 0.375 and 0.650, which is the sign of the low degree of fixity of the ends.

In trying to account for this difference, we found that it was due solely to the faulty centering of the specimens in the second group in relation to the compressing load. On the other hand, in the first group we proved that the centering was practically perfect. In the second group, however, the errors in centering were very slight, about 2 mm. We therefore see that a very slight error in centering, which it is very difficult to avoid in practice, is sufficient to destroy in a large measure the effect of the fixity of the ends.¹

In actual structure the eccentricity of the loading is nearly always much greater than 2 mm., and the ends of these pieces are never held as rigidly as were those in our tests; in these conditions the effect of the fixity of the ends must necessarily be very small in almost all cases, and for considerations of safety we must neglect it in our calculations. So we may say that in this respect there is no difference between welded and riveted structures.

Secondary Stresses at the Knots.—This question is very closely allied with the preceding one and was very thoroughly studied for riveted structures by the Swiss Commission mentioned. The following are some of its conclusions:

1. The gusset pieces and other assembling pieces are subjected to evident and unquestionable elastic deformations.

2. Generally speaking, the stiffer the structure the less the secondary strain.

¹In order that this description might not be unnecessarily long and since in this case the weld itself is not concerned, we did not think it expedient to reproduce here all the figures and calculations of these special tests.

3. In structures of normal design and in bars of usual slenderness, secondary stresses in the neighborhood of the knots are not more than 15 to 20 per cent of the admissible loads.¹

How can these conclusions be applied to welded assemblings? The gusset plates are very small and light, and will therefore stretch a great deal relatively, thereby reducing the secondary stresses. A welded structure is stiffer as a whole than a riveted one. The members in a welded structure, whose ends are not weakened by rivet holes, will as a rule have a higher ratio of slenderness than those in a riveted structure, and we know that in welded structures the rational design of the knots is easier owing to the absence of the cumbersome rivets.

From all this it necessarily follows that in a welded structure the secondary stresses will be less than in the corresponding riveted structure. This conclusion is worthy of particular note, for the opposite opinion has frequently been advanced.

We now know exactly how the joints of a welded structure will behave under static loads. It remains for us to see how they will behave under dynamic loads and repeated loads.

Dynamic Loading.—Our tests have shown how great was the strength of side welds under dynamic loading, and how quickly this strength increased with the length of the weld.²

If by means of side welds exclusively we assemble plates of 40-kg. steel so that the resistance of the joint to static loads is equal to that of the welded parts, we know that the strength of the joint under dynamic loading will be greater than that of the parts. This will be true *a fortiori* in the case of shapes, for the welds will be proportionately longer than in the case of plates.

Side welds, then, provide us with a very easy method of making assemblings which will be at least as strong as the parts welded under dynamic loading. This is very important and worthy of special note, for many engineers think that since welds do not behave very well under the Charpy test, welded structures

¹ Most of these stresses have been computed from the elongation of the various fibers, measured on very short lengths (10 to 20 mm.) close up to the knot by high precision extensometers (Zweiss, Huggenberger, etc.). The commission demonstrated that this was the only way in which they could be accurately determined. Except in very simple structures the different methods suggested for calculating these secondary stresses gave very different results from those actually observed.

² It appears to vary approximately with the square of the length.

must also be weak when subjected to impact stresses. This is quite wrong.

Repeated Stresses.—We have seen that our electrode deposits a metal with a very clearly defined “endurance limit” under alternate stresses at 15 kg. per square millimeter. Under repeated but not reversed stresses (*i.e.*, varying between zero and a maximum), we know from Wohler, from American tests, etc., that the “endurance limit” will be around $1\frac{1}{2}$ by 15 kg. = $22\frac{1}{2}$ kg.

We did not carry out fatigue tests in shear, but we know that the endurance limit in shear may be taken as equal to half the corresponding limit in tension.¹ So we can assume $7\frac{1}{2}$ kg. for alternating stresses and $11\frac{1}{4}$ kg. for stresses varying from zero to a maximum.

What does this mean applied to welded structures? In order to produce in the dangerous section of a front weld stresses reaching the endurance limit (*i.e.*, 15 or $22\frac{1}{2}$ kg.), the metal of the assembled parts must be stressed to $15 \times \frac{\sqrt{2}}{2}$ or $22\frac{1}{2} \times \frac{\sqrt{2}}{2}$, *i.e.*, $11\frac{1}{2}$ or $15\frac{1}{2}$ kg. per square millimeter.

In order to produce in the dangerous section of a side weld stresses reaching the endurance limit (*i.e.*, $7\frac{1}{2}$ or $11\frac{1}{4}$ kg.), the metal of the assembled parts must be stressed to 5, 3 or 8 kg. per square millimeter.

Usually under static loads tensile stresses are limited to 10 kg. per square millimeter. This limit is often reduced to 8 kg. for parts subjected to rapid varying loads like cross-girders in railway bridges; exceptionally the limit is raised to 12 kg. for parts where the permanent loads are preponderant. A comparison of these figures shows clearly that:

(a) Front welds, calculated in the way we have recommended, will never in practice be strained to their endurance limit. In other words, in front welds the effects of fatigue can safely be neglected.

(b) Side welds, calculated in the way we have recommended, should in certain cases be reinforced on account of the effects of fatigue.

This requires some explanation. Such strengthening is only necessary if we wish to have a structure equally safe in all parts,

¹ Illinois University, 1923, 19 steels. For 2 of these the relation of the two endurance limits was 0.49; for the other 17, 0.50 or more. The average was 0.534.

and in this we are demanding much more than in the case of riveted structures.

It is well known that rivets are calculated in a purely empirical manner. It is assumed that the ultimate strength in shear of the metal of the rivet is equal to 80 per cent of its ultimate strength in tension and that the elastic limits, the elastic modulus, and the endurance limits are in the same proportion. It is not very long since it was discovered, but quite definitely, that the actual relation between the ultimate strengths is only 70 to 75 per cent and that the relation between the elastic limits and endurance limits (the factors of the greatest importance in the case under consideration) is always around 50 per cent. So the real factor of safety against repeated stresses in many riveted joints is much lower than one would expect. As a rule side welds should be reinforced by lengthening.

We must here repeat that it would be dangerous, especially in the case of repeated stresses, to try to reinforce the joint by using a stronger electrode, unless the metal of the welded part is itself of high quality.

From the foregoing it will be seen that the current belief that the resistance of welded joints to repeated stresses is rather low is without foundation.

It is easy to make welded joints as strong against repeated stresses as the pieces welded.

Welded structures, calculated in the way we have recommended, will have a higher factor of safety than that of most existing riveted structures.

We have now examined all the points which normally must be taken into consideration when calculating a metallic structure.

Summary of Conclusions.—The principal conclusions resulting from our study may be summarized as follows:

1. Structural steels may be butt welded, lap welded, or gusset welded in such a way that full use is made of the strength of their entire section.

2. It is easy to assemble shapes by welds so that the resistance of the resulting structure against static loads is equal to that of the shapes themselves.

3. With regard to the long compressed members, a welded structure behaves exactly in the same way as a riveted structure. The effects of the fixity of the ends are negligible.

4. "Secondary stresses" which develop in proximity to the knots are slightly less in welded structures than in riveted structures. In a well-designed structure they are never very important.

5. It is easy by employing "side welds" to make a welded joint as strong against dynamic stresses as the welded parts.

6. It is easy to make a welded joint as strong against repeated stresses as the welded parts. In the case of repeated stresses, in the interests of economy front welds must first be used.

7. It is easy by following the foregoing recommendations to make a structure in which all parts shall have the same factor of safety; *i.e.*, structures in which the joints shall be just as strong as the assembled parts themselves against all kinds of stresses.

8. Of all structures having the same form and the same factor of safety, the welded structure calculated according to our recommendations is the most economical possible.

PART III

ELECTRIC WELDING OF SHIPS' BULKHEADS AND
SIMILAR PLATED STRUCTURES

BY

COMMANDER H. E. ROSSELL

CHAPTER I

PRELIMINARY DISCUSSION

General.—Much has been written on the application of the electric-welding process to ship-hull construction, but as a rule the writers have dealt in generalities and have failed to give sufficient design information to enable the naval architect to attack the problem with confidence. Consequently, very little real progress has been made in the use of welding as a recognized method of jointing on the main structural or strength parts of ships.

The purpose of this paper is to deal with the subject of structural welding particularly as applied to ships' bulkheads. The data and discussion are applicable also to plated structures such as gas, oil, and water tanks.

It appears to the author that the most important thing to determine in connection with this problem is the behavior of the deposited metal when the structure is loaded to destruction. The properties of welded test specimens have been investigated by many experimenters, and the results of these investigations have been published and are available for such use as designers may care to make of them. The behavior of the deposited metal, when worked into a completed structure, is still comparatively unknown, however; and the designer is forced therefore to rely heavily on his own judgment. The use of judgment unassisted by a background of known facts and experience is so surrounded by pitfalls that it is not attractive to the conservative designer.

Properties of Electric Welds.—Although, as has been stated, the properties of metal deposited by arc welding are well known, it is desirable for the sake of clearness and completeness to outline them briefly.

In the present state of the art, the deposited metal lacks uniformity in strength, and it is necessary therefore to use conservative strength figures in designing. The author recommends the use of 36,000 lb. per square inch for the ultimate

tensile strength of butt welds (Fig. 1), and for the ultimate strength in tension and shear of lap welds, loaded as indicated in Fig. 2. Lap welds loaded in the direction of the lines of welding (Fig. 3) are slightly weaker than those loaded normal to these lines. In the design of the former, the use of an ultimate strength in shear of 29,000 lb. per square inch is recommended. The foregoing unit-strength figures apply to section A-A in each case. The strength of lap welds falls off with an increase in the thickness of plating welded, but the decrease is not material within the range of thicknesses usually employed in ship-bulkhead construction; *i.e.*, up to a limit of $\frac{3}{4}$ in.¹

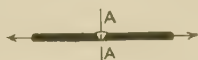


FIG. 1.—Butt joint.

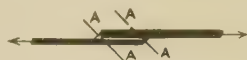


FIG. 2.—Lap joint loaded normal to lines of welding.

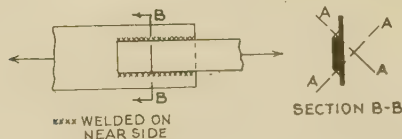


FIG. 3.—Lap joint loaded parallel to lines of welding.

In general, lap joints should be welded along both edges and the total length of weld in both lines should be considered as contributing to the strength.

The ultimate strength in shear of sound electric welds is approximately 35,000 lb. per square inch of cross-section, but it appears desirable to use in design a somewhat lower figure, say 29,000 lb. per square inch.²

The modulus of elasticity of the deposited metal is approximately the same as that of structural steel. As the former lacks uniformity in strength, however, its elastic limit is frequently nearly as great as its ultimate tensile strength. Beyond the elastic limit the deposited metal is very much less ductile than

¹ MELLER, K.: "Electric Arc Welding;" OWENS, JAMES W.: "Welding Research Activities of the Newport News Shipbuilding and Dry Dock Company During 1926," annual meeting American Welding Society, 1927; DRESE, E. E., "Evolution in Construction," *The Welding Engineer*. Tests conducted at Norfolk and other United States Navy Yards, 1912-1927; HOEHN, E., "The Strength of Electric Welded Hollow Bodies."

² Report on longitudinal shearing tests of rail plate welds, *Jl. Am. Weld. Soc.*, March, 1927.

structural steel similarly stressed. This naturally leads to the suspicion that welded joints in highly stressed structures may have a tendency to act as "hard spots."

Metal deposited by the arc-welding process is low in resistance to bending, and its properties in this respect are subject to wide variations and depend largely on the skill of the welder. The average angle through which welded specimens can be bent without fracture is only about a third of that which can be withstood by structural-steel specimens similarly tested. Moreover, a line of welding applied to a structural-steel member will reduce the resistance to bending of the latter. The amount of this reduction has not been determined definitely, but it is greater for a bend away from the side on which the line of welding is applied than for one toward that side. The reduction in resistance to bending is considerably less for a specimen that has had the bead of weld ground off than for an unground specimen.

Another objectionable effect¹ of applying welding to structural-steel members has been investigated. The heat of the arc fuses part of the steel member, and the fused metal subsequently solidifies as a cast structure. The material below the fused portion is heat-treated with the formation of sorbite, some troostite, and with a refinement of the grain. The tensile strength of the metal adjacent to the line of welding is not affected adversely, but its resistances to shock and fatigue are probably lowered appreciably.²

The resistance to shock of electric welds is far below that of structural steel. Impact tests on unwelded and on welded mild steel specimens, conducted in 1923 by the Swiss Association of Steam Boiler Owners,³ gave mean values for impact energy at failure about one-third to one-sixth as high for arc-welded bars as for unannealed steel bars. Moreover, the welded bars were

¹ PIERCE, Lieut. (jg) H. W. (CC), U. S. N., and Lieut. (jg) DALE QUARTON (CC), U. S. N., "The effect on the strength of a continuous ship member of arc welding to it an intercostal piece," thesis for degree of master of science at Massachusetts Institute of Technology, 1926. Tests at Norfolk Navy Yard, 1925.

² Since this was written, tests made at the Naval Experimental Station, Annapolis, Md., have indicated that lines of welding across medium- and high-carbon steel members do not lower the tensile strengths or endurance limits of these members.

³ HOEHN, E., "The Strength of Electric Welded Hollow Bodies."

not at all uniform in their resistance to shock. In fairness to the welded specimens it should be added that the material of the bars was very ductile, showing an elongation in about 8 in. of 30 to 40 per cent and a reduction in area of 60 to 70 per cent.

Information available as to the endurance limit of welded joints against fatigue is not great. The latest paper on this subject known to the author was read by R. R. Moore¹ at the April, 1927, meeting of the American Welding Society. An endurance limit of 9,000 lb. per sq. in. was given for a "cleaned up" arc-welded tube, but the author of the paper stated that this specimen was found to contain a flaw in the weld. Endurance limits of 14,000 to 16,000 lb. per square inch were given for larger tubular specimens that had not been "cleaned up." Fatigue tests conducted some years ago by Lloyd's Register of Shipping gave an endurance limit of about 14,500 lb. per square inch for welded bars and a limit of about 23,000 lb. per square inch for unwelded bars.²

Objections to Riveted Joints.—The foregoing discussion may seem like a severe arraignment of the qualities of arc-welded joints. Before hasty conclusions are drawn, however, consideration should be given to the relatively low unit stresses recommended for use in the design of such joints. It should also be noted that riveted joints in plated structures are far from satisfactory. Their strength varies from about 40 to about 75 per cent of that of the parts connected, and the higher strengths are obtained only at the expense of providing several rows of rivets in each joint.

At relatively low stresses the rivets are subject to slip,³ with resulting leakage. This feature is extremely objectionable in ship bulkhead construction, especially on bulkheads forming the boundaries of oil or water tanks.

An attachment made by riveting to a passing member weakens the latter in tension roughly in proportion to the amount of metal removed for rivet holds. This condition may be considered as somewhat analogous to the effect of lines of welding

¹ "Fatigue of Welds."

² Cox, H. H., "Lloyd's Experiments on Electrically Welded Joints," *Gen. Elec. Rev.*, December, 1918.

³ "An Investigation of the Behavior and of the Ultimate Strength of Riveted Joints Under Load," by GAYHART, E. L., *Trans. Soc. Naval Arch. and Mar. Engrs.*, 1926.

on passing members, although the welding probably affects the shock and fatigue resistances of these members rather than their tensile strength.¹

Another bad feature of riveting that usually is not mentioned, because it is so old that we are used to it, is the necessity for adopting very complicated, expensive, and heavy designs of joints to accommodate the rivets. Still another objection lies in the greater rate of corrosion for rivets than for the surrounding plates and shapes. The working that the rivets get in driving, results in a change in their electric potential, which is often followed by rapid pitting of the rivet points in service. Metal deposited by welding is not subject to this accelerated corrosion.

In view of the many objections to riveting it may be said with little fear of contradiction that the naval architect is not unfavorably disposed toward the substitution of electric welding as a jointing method in ship construction, provided he can be convinced that it is as cheap, strong, and reliable as riveting and provided that he feels sure of the way the deposited metal will behave in the completed structure.

Functions and Loading of Ships' Bulkheads.—Before considering in detail the problems involved in the use of welding on ships' bulkheads, it appears desirable to outline briefly their functions and the various ways in which they are loaded.

A ship is essentially a hollow girder subdivided and stiffened at intervals by transverse and longitudinal partitions or bulkheads and by horizontal diaphragms or decks. The ship in a seaway may be visualized as being supported alternately on the crest of a wave at her midship section and on two crests at or near each end of the ship. Under such conditions the upper and lower continuous longitudinal portions of the structure act alternately in compression and in tension. The bulkheads stiffen the ship girder and hold it up to its work. The transverse bulkheads also resist racking—that is, the action tending to distort them into a diamond shape.

Both longitudinal and transverse bulkheads are called upon to take compressive loads under various service conditions. These loads originate in the weight of the component parts of

¹Since this was written, tests made at the Naval Experimental Station, Annapolis, Md., have indicated that lines of welding across medium- and high-carbon steel members do not lower the tensile strengths or endurance limits of these members.

the ship or conversely in the upward forces of buoyancy. As a rule the most severe compressive loads on bulkheads occur during drydocking or during grounding of the vessel. Transverse bulkheads, particularly those near the bow of a ship, must also withstand compressive loads imposed by the dynamic action of the sea.

If the shell of the ship is broken through below the water line, the adjacent bulkheads serve to restrict the entry of water into the interior. It is under such conditions that most bulkheads may be expected to withstand their most severe loads, and it is customary to design them to take such loads without deflections so great as to make the bulkheads incapable of fulfilling their other functions. There are of course exceptional cases, such as bulkheads acting as gun foundations, where loads other than hydrostatic pressure are the determining considerations in the design. In general, however, a bulkhead which is capable of withstanding, without excessive deflection, the pressure to which it may be subjected under service or emergency conditions may be assumed to have sufficient strength for all normal conditions of loading.

Bulkheads forming the boundaries of oil or water tanks are required to resist pressure under ordinary operating conditions, and they are designed somewhat more conservatively than bulkheads that will be subjected to pressure only under emergency conditions.

It is obvious that transverse bulkheads act as separate structures rather than as parts of the ship girder. The same holds true for longitudinal bulkheads of limited extent and which are not directly attached to longitudinal strength members of the ship.

It appears reasonable therefore completely to weld such bulkheads in a ship otherwise riveted, without fear that the welded joints of the bulkheads will have to take unknown additional stresses owing to the slip in nearby riveted joints.

CHAPTER II

OUTLINE OF BULKHEAD TESTS

Purposes.—Having in mind the desirability of securing comprehensive information as to the possibilities of welding on plated structures such as ships' bulkheads, the Bureau of Construction and Repair of the Navy Department recently authorized, on the recommendation of the author, a series of hydrostatic tests to destruction on fairly large bulkheads, 12 ft. by 12 ft., all but two of which were completely welded. Two of the bulkheads were riveted and were intended to set a standard of comparison for the remaining ones. The purposes of the tests may be summarized as follows:

(a) To determine how deposited metal behaves when worked into plated structures which are loaded to destruction.

(b) To determine whether ultimate strength values of the order of those given earlier in this paper may be used with confidence in the design of the joints of such structures or whether these figures should be reduced to take account of additional stresses coming on the welded joints after part of the other material of the structure has passed the elastic limit; in other words, to determine whether the welds tend to act as "hard spots."

(c) To develop a thoroughly practical design for welded bulkheads which will save a maximum of weight without sacrifice of strength or stiffness, which will permit of easy and cheap shop assembly, and which will require a minimum of field welding during erection.

(d) To test out certain details of welding design making for lightness and cheapness, such as welded collars (Fig. 5) instead of smithed staples (Fig. 4), welded pipe connections instead of cast bolted ones, and welded plate boundaries instead of bounding angles.

(e) To develop the technic of shop assembly with a view to determining the feasibility of the use of automatic welding machines on this work; to devise means for preventing warping

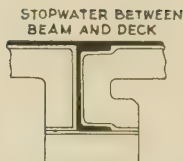


Fig. 4.—Typical stapling around beam piercing riveted bulkhead.

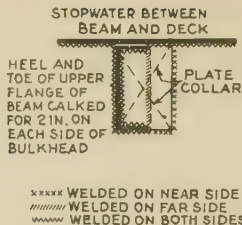


Fig. 5.—Typical plate collar around beam piercing welded bulkhead.

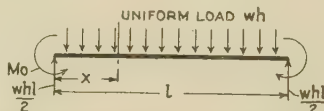
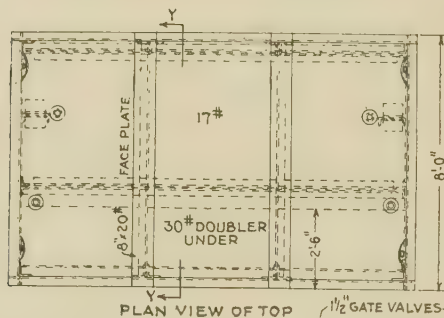


Fig. 6.—Uniformly loaded beam fixed or partially fixed at ends.



-NOTES-

- = DEFLECTION STATION
- + = STRAINMETER STATION
- WELDING THICKNESSES ARE SHOWN IN CIRCLES, THUS (1/2), AND INDICATE THE LEG LENGTH OF FILLET IN INCHES
- WHERE MARKED THUS (R) THE LETTER "R" MEANS REINFORCED

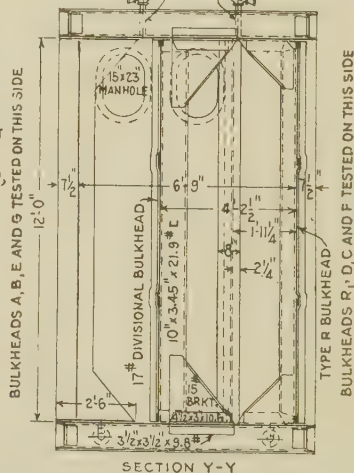


Fig. 7.—Test tank.

during assembly; to develop an assembly procedure requiring only flat welding and calling for a minimum of handling of material.

(f) To develop the technic of field erection, having in mind the use frequently made of bulkheads to hold the ship in shape during construction.

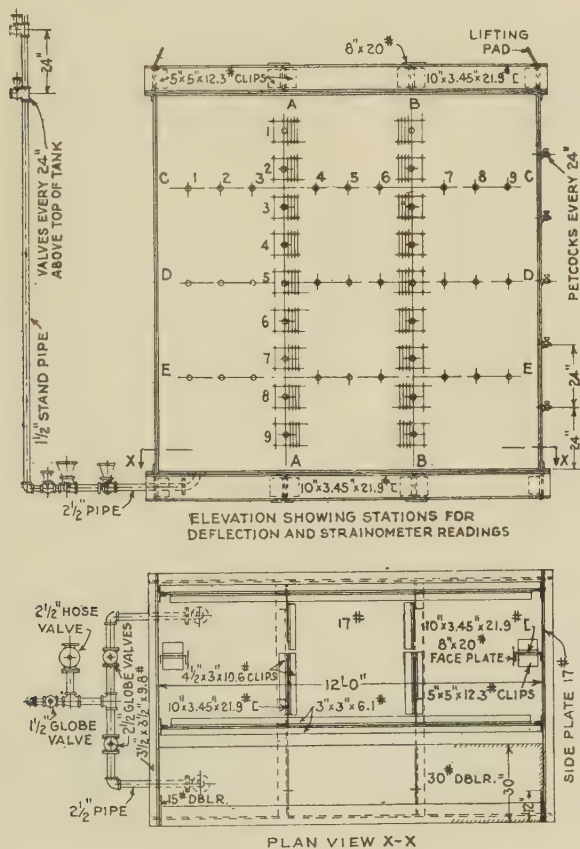


FIG. 8.—Test tank.

(g) To ascertain weight and cost savings with equal strength which can be made by welding bulkheads instead of riveting them.

Designs of Test Tank and Bulkheads.—A large test tank, 12 ft. wide by 12 ft. high by 8 ft. long, with heavily reinforced top and bottom, was constructed for the purposes of mounting

the bulkheads to be tested and of putting progressively increasing hydrostatic pressures on them. The tank was fitted with a very heavy internal bulkhead dividing it into two compartments, each of which was provided with a vent and a standpipe. The test head in either compartment could be reduced quickly

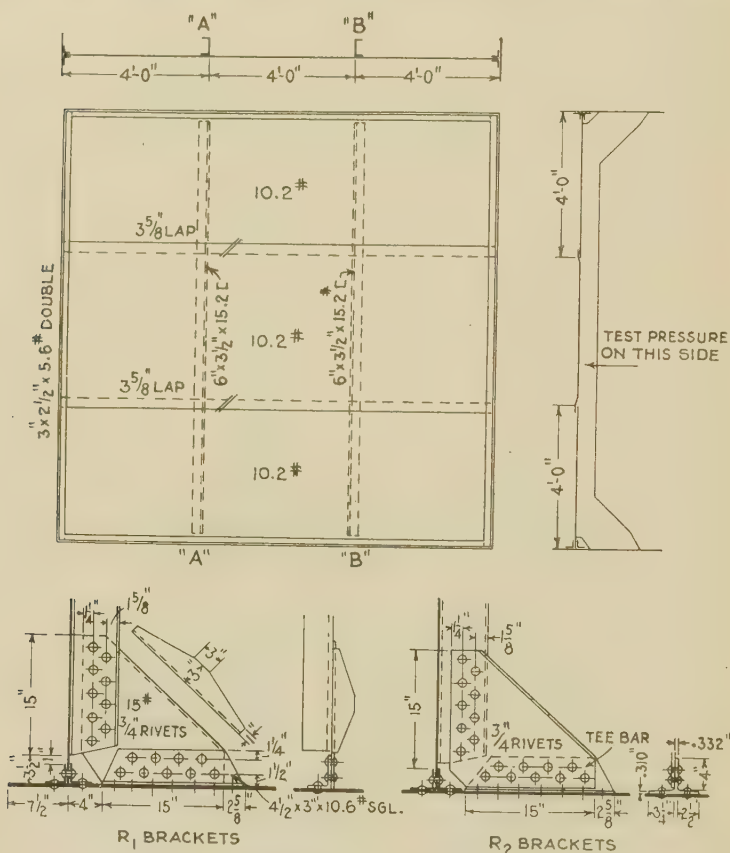


FIG. 9.—Riveted bulkheads types R_1 and R_2 .

to the level of the top of the tank by draining its standpipe. The design of the tank is shown in Figs. 7 and 8.

The designs of the test bulkheads are given in Figs. 9 to 16. These bulkheads may be described briefly as follows:

Bulkhead R_1 .—A completely riveted bulkhead, having three horizontal strakes of 10.2-lb. plating, seams being joggled and double riveted with 5/8-in. pan head, countersunk-point rivets

spaced four diameters; 6- by 3½- by 3½-in., 15.3-lb. vertical channel stiffeners, spaced 4 ft. apart, were riveted to the plating by ⅝-in. pan head, countersunk-point rivets, spaced six diameters. Flanged brackets, about 18 by 18 in., 15 lb., were fitted

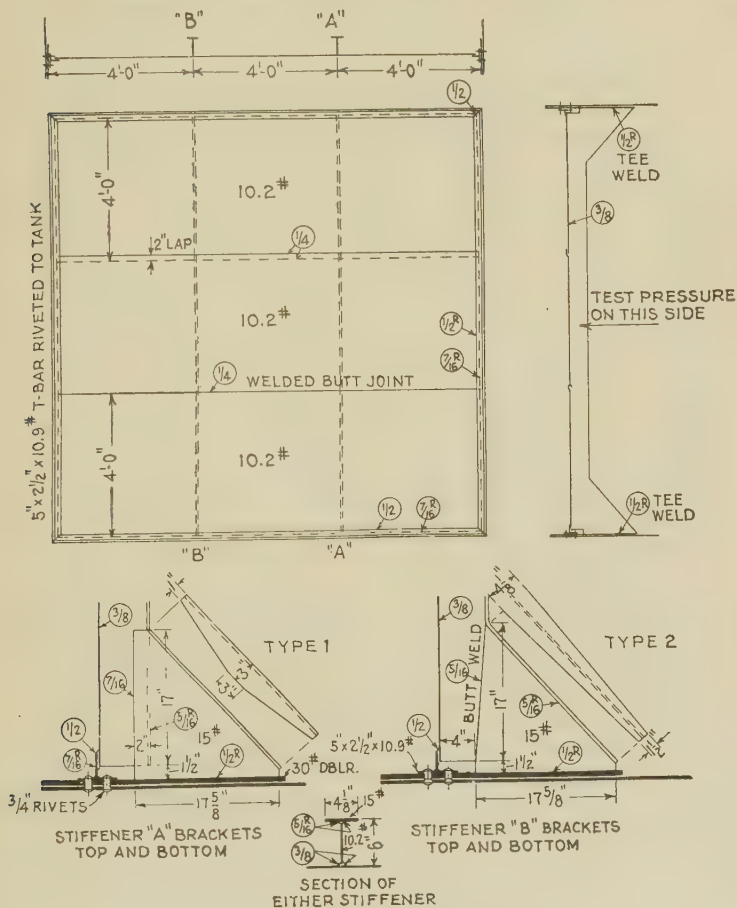


FIG. 10.—Welded bulkhead type A.

at the top and bottom of each stiffener, a 4½-by 3-in., 10.6-lb. angle clip connecting each bracket to the tank. Double bounding angles 3 by 2½ in., by 5.6 lb., were fitted around the bulkhead.

Bulkhead A.—A completely welded bulkhead having three horizontal strakes of 10.2-lb. plating. The lower seam was butt welded and the upper seam was joggled and lap welded. The

mittent), as well as those connecting stiffener face plates to webs; but it was decided that this would not prevent the welds from taking tension or compression, and might make them more liable to failure owing to their discontinuity in strength. Stiff-

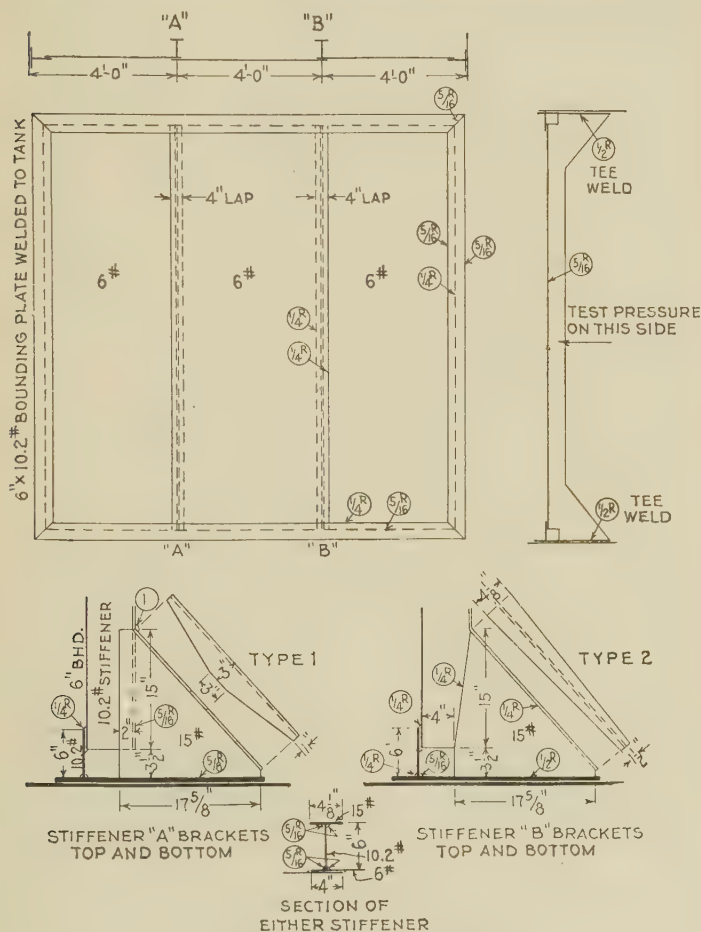


FIG. 12.—Welded bulkhead type C.

eners were fitted with 15-lb. brackets of the same size as those on bulkhead R₁, but connected to the test tank by welding instead of riveting.

On one stiffener type 1 brackets were fitted, while on the other type 2 brackets were used. Both types are detailed in

where welded boundary connections might not be considered permissible.

Bulkhead B.—A completely welded bulkhead having three vertical strakes of 7.65-lb. plating connected by lap joints. These joints were so located that they would contribute to the

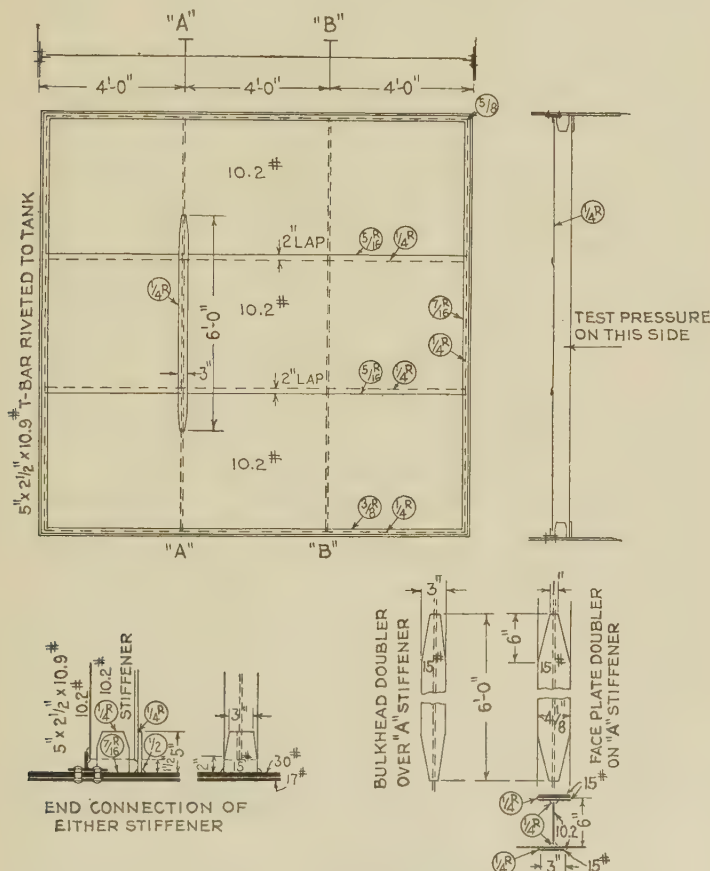


FIG. 14.—Welded bulkhead type E.

section modulus of the stiffeners. The same spacing and the same general design of stiffeners as for bulkhead A were used. Stiffeners were designed with the same section modulus as for bulkhead A. The bulkhead plating was connected to the tank by means of a 6-in. strip of plating, tee welded to the tank and lap welded to the plating.

as shown in Fig. 14 and except that both plate seams were lap welded. One stiffener was fitted with face plates, as indicated, in order to increase the section modulus to take the increased bending moment resulting from the omission of the brackets.

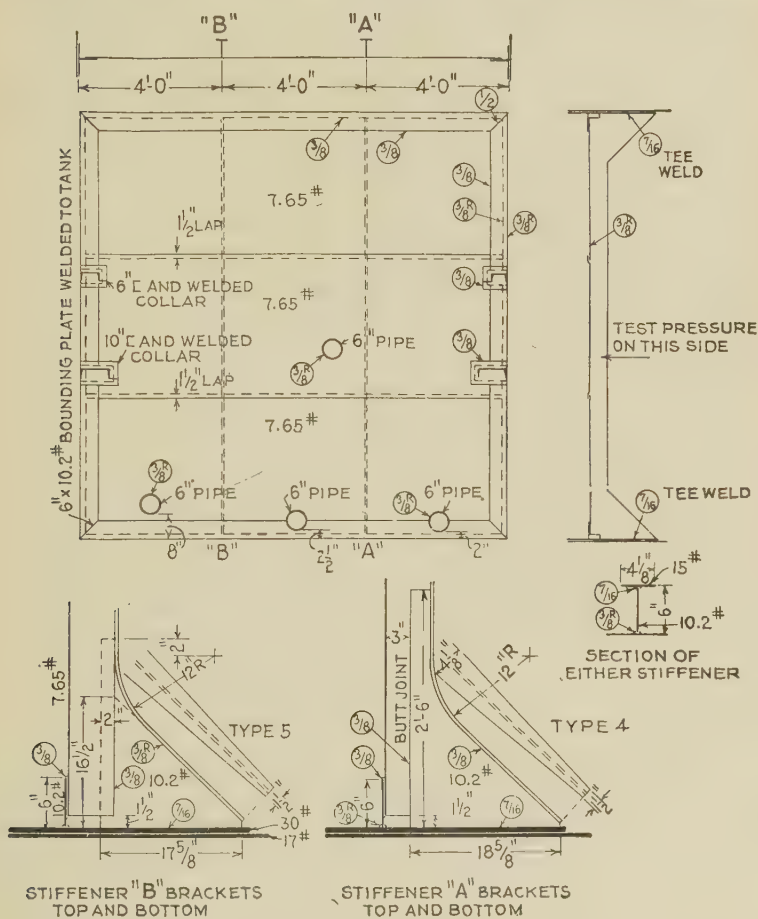


FIG. 16.—Welded bulkhead type G.

The other stiffener was not fitted with the extra face plates, and greater deflections and earlier failure in this stiffener were expected.

Bulkhead F.—Same as bulkhead B except that stiffeners were fitted on the outside rather than on the tank side. One of the stiffeners had brackets of type 2, while the other had similar

brackets except for the shape at the apex, where an easy curve was used instead of a knuckle. Modified brackets, called type 3, are shown in Fig. 15.

Bulkhead G.—Similar to bulkhead A except that plating was 7.65 lb. and brackets were of 10.2-lb. plating and of modified design, as shown in Fig. 16. Plate boundaries were used. Two sections of 10-in. channel and two of 6-in. channel were installed through the bulkhead at its sides, welded plate collars being fitted for tightness. Four sections of pipe, each plugged at one end, were rove through the bulkhead and welded thereto.

Bulkhead R₂.—The same as R₁ except that tee- instead of angle-bar bracket clips were used. There were two rows of rivets securing each clip to the tank.

Deflection and Strain Measurements.—Arrangements were made for the taking of a rather elaborate set of deflection and strain measurements. Vertical piano wires were stretched parallel to each stiffener, and similar horizontal wires were provided at intervals of 3 ft. Strain-gage readings were taken along and on either side of each stiffener. The layout of wires and gage points is given in Fig. 8. Deflections were measured by means of a rule with a hardened pointer attached to its end. In taking a measurement the pointer was inserted in a center punch hole on the bulkhead and the scale of the rule read where it was crossed by the wire. Wires were kept taut by weights working over small pulleys suitably located. The deflection readings were taken to the nearest twentieth of an inch and were considered accurate to within $\frac{1}{10}$ in.

Three Berry strain gages with 8-in. base were used. One of the gages, which was used to take all readings on stiffener A of bulkheads R₁ and A, was injured before it could be calibrated. The other two gages, which were employed for the remainder of the readings, were calibrated by the Bureau of Standards. Gage holes were made with a size 55 drill operated through a special jig which had been furnished with the gages. The holes were later reamed by a hand tool to remove any irregularities. All gage readings were repeated until a check was obtained.

Distortion of Test Tank.—Distortion of the sides of the tank was checked by means of four plumb bobs hung at its four corners, as shown in Fig. 34. During tests of bulkheads R₁ and A, the sides of the tank at its ends did not deflect in more than $\frac{1}{2}$ in. For the test of bulkhead B and also for that of bulkhead

D (the fourth one tested), up to failure of the stiffeners, the deflection was as great as $\frac{3}{4}$ in. After the stiffeners of bulkhead D had failed, the sides of the tank deflected in about 3 in. For subsequent tests much of this distortion remained in the tank, the bulkheads being cut to the shape of the deformed tank prior to installation. During these tests further inward deflections of the order of $\frac{1}{2}$ in. were noted. Of course the deflections mentioned were in excess of those abreast the bulkheads. A straight-edge placed along the stiffeners on the top of the tank showed that these members remained practically straight throughout the tests.

Specific Gravity of Water.—The specific gravity of the water used in the tests varied from 0.9775 to 0.9825, and averaged 0.98.

Test Heads.—It was planned to apply water heads to each bulkhead in 2-ft. steps and to return to a 12-ft. head after application of heads of 20, 28, 36, and 44 ft. All deflection readings and strain-gage readings for points in line with each stiffener were to be taken before and after each test and at each head applied during the test. Complete strain-gage readings were to be taken before and after each test, at each 12-ft. head and at intervals of 8-ft. above the 12-ft. head. This program was followed in general, but it was curtailed or extended in individual cases as dictated by circumstances. The maximum head used in any case was 66 ft., but this head did not cause failure of several of the bulkheads.

CHAPTER III

CONSTRUCTION OF BULKHEADS

Shop Assembly.—Riveted bulkheads R_1 and R_2 were fabricated and riveted in the shop, hand pneumatic hammers being used. They were installed in the tank in accordance with methods in common use in all shipyards. The bounding bars were calked after installation of the bulkheads in the tank. About 20 per cent of the rivets showed minor leaks under pre-

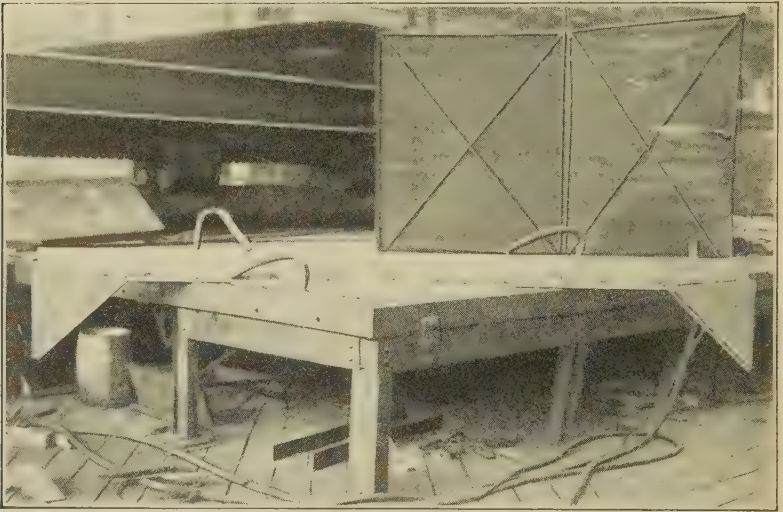


FIG. 17.—Stiffener with type-2 brackets (used in bulkheads A, B, C, D, and F) under construction.

The pieces of this stiffener have been tacked in place and the stiffener is dogged down preparatory to welding.

liminary water pressure, and these leaks were stopped by calking the rivets concerned prior to the application of test pressures.

As the designs of the welded bulkheads had been worked out with a view to having almost all the welding done in the shop with the work in a flat position on the floor, it was considered proper to take full advantage of this design feature during their construction. The methods used in building the welded bulk-



FIG. 18.—Stiffener with type 3 brackets in process of welding.
Note curve in face instead of sharp break.



FIG. 19.—Type F bulkhead tack-welded and dogged down for final welding.

heads are clearly illustrated in Figs. 17, 18, 19, and 20. The plating was first sheared to size and then laid out on the slab and dogged down securely. For large-sized bulkheads it would probably be necessary to use more elaborate clamping devices to hold in a plane the center portion of the plating. The laps were then tack welded preparatory to the running of a complete weld down the edge of each lap. It is of interest to note the ease with which automatic welding machines could be adapted to this and also to later operations. The sheet of plating was next turned over, re-dogged down, and welded along the edges of the plate laps. It was found that warping of the plating was minimized if the welds along the laps were run continuously from the same end of the bulkhead. On the first bulkhead welded, A, this procedure was not followed, and the plating was considerably and somewhat irregularly distorted.

Generally similar methods were followed in the construction of the stiffeners. When the brackets and face plates had been attached to the webs of the stiffeners, the completed assemblies were tack welded to the plating. The continuous tee welds between the plating and the webs were made by "stepping back" or moving the arc frequently from place to place in an attempt to keep the temperature as nearly constant as practicable throughout the length of the stiffeners.

Measurements of the deflections of the various bulkheads as installed in the tank and before the application of pressure are given in Table 1.

Installation of Bulkheads.—The installation of each of the welded bulkheads in the tank was accomplished without difficulty by shoring the bulkhead against its boundary bar or plate which had been previously installed in the tank. The ease with which this work was done demonstrated the practicability of the two designs of boundary connections used. Some trouble had been expected in performing the welding of the T boundary bars to bulkheads A and E owing to the limited space available for the welder, but it was found that even this part of the work could be done with facility. Figures 34, 35, and 36 give views of the second design of boundary connections used. By means of boundaries such as these most of the difficulty in getting bulkheads to fit in place on a ship could be eliminated. In case a bulkhead were to be used to assist in drawing the surrounding structure into its proper shape, the plating of the former could

TABLE 1.—EXTENT OF WARPING DURING ASSEMBLY AND ERECTION
(Maximum deflection in inches along lines indicated)

Bulk-head	Description	A stiff-ener	B stiff-ener	C plating	D plating	E plating
R ₁	10.2 lb., riveted.....	+ .15	+ .10	+ .15	+ .10	+ .10
		— .15	— .10	0	— .20	— .05
A	10.2 lb., welded.....	+ .25	+ .20	+ .65	+ .65	+ .45
		— .25	— .05	0	— .15	— .10
B	7.65 lb., welded.....	+ .35	+ .25	+ .55	+ .65	+ .65
		— .15	— .10			
C	6 lb., welded.....	+ .30	0	+ .70	+ .90	+ .65
			— .35			— .20
D	6 lb., welded.....	+ .10	+ .30	+ 1.00	+ .30	+ .95
		— .25	0			
E	10.2 lb., welded.....	+ .20	+ .20	+ .55	+ .50	+ .40
		— .10	— .10	— .15	— .20	— .40
F	7.65 lb., welded.....	+ .30	+ .25	+ .65	+ .25	+ .50
		— .15	— .25		— .50	
G	7.65 lb., welded.....	+ .20	+ .30	+ .10		+ .20
		— .55	— .25	— .45	— .65	— .55
R ₂	10.2 lb., riveted.....	Not measured; probably of same order as for R ₁				

NOTE.—Measurements were made, prior to test, from a plane parallel to that of the piano wires along lines *A-B-C-D-E* of Fig. 8. Positive sign signified deflection toward stiffener side; negative sign, the reverse.

be welded to the correct size of the completed bulkhead. The surrounding structure could then be pulled up to the bulkhead plating by means of angle clips and bolts preparatory to tee welding the edges of the bulkhead directly to this structure, no boundary bars or plates being used.

Workmanship.—In the construction of the test bulkheads no attempt was made to pick workmen above or below the average in skill. Inspection and supervision were rigid, however, and the interest aroused by the tests among the personnel of the building yard was keen. As a result the workmanship was good; in fact, that employed in driving the rivets of bulkheads R₁ and R₂ was distinctly above average.

The following quotation from the report of the Commandant of the Norfolk Navy Yard, which built the bulkheads, is of interest in connection with the quality of welding:

It cannot be too strongly emphasized that the standard of welding obtained on these bulkheads was no better and no worse than that which would be obtained on new construction under ordinary conditions. The following men (there were 23 first-class welders carried on the rolls at the time of the tests) took part in the work at one time or other:

Welders rated No. 1,* 3,* 9,* 10,* 17,* and 22 on the first-class list; one second-class welder; one third-class apprentice welder;* one fourth-class apprentice welder. The six men whose names have been marked with an asterisk (*) worked on all seven of the welded bulkheads.

Two and in some cases three men worked on the bulkheads at one time. All work of welding was done in at least two shifts, and at the end three shifts were employed. It may readily be seen that a total of approximately 550 hr. of welding, a good part of which was carried out on the inside of a closed compartment, done by men as widely scattered on the efficiency list and under the conditions outlined, cannot in any sense be considered other than ordinary work.

Direct-current metal-arc machines were used for all welding. Data as to current values, sizes of electrodes, amount of welding, and qualification tests of welders are given in Appendix A. The leg lengths of welds (thicknesses for butt welds) as actually made were measured, and the measurements are indicated in Figs. 10 to 16, by figures in circles. Letter "R" in a circle means that the weld in question was reinforced.

Material.—The material used in the construction of the bulkheads was analyzed chemically and tested physically, the results being given in Appendix B. The high carbon content of some of the material is of interest.

Weight and Cost Data.—Accurate weight and cost data were taken, and they are given in Table 2. The weights include the bulkheads complete with boundary connections. The costs are expressed in percentages of the cost of riveted bulkhead R_1 and include all labor, material, and overhead expense involved in the building and installation of the bulkheads. Cost data were also taken for riveted bulkhead R_2 , but are not included in the table as the work was done under conditions different from those existing when the rest of the bulkheads were built.

TABLE 2

Bulk-head	Description	Weight, pounds	Weight, percent-age of R ₁	Cost, percent-age of R ₁
R ₁	10.2 lb., riveted.....	2,600 ¹	100	100
A	10.2 lb., welded.....	2,270 ¹	87	86
B	7.65 lb., welded.....	1,550	60	72
C	6 lb., welded.....	1,350	52	69
D	6 lb., welded; pressed panels.....	1,350	52	71
E	10.2 lb., welded; no brackets.....	2,240	86	86
F	7.65 lb., welded.....	1,560	60	78
G	7.65 lb., welded.....	1,580 ²	61	81

NOTE.—Where mold loft work was unnecessary because of suitable molds being available from bulkheads, built previously, adjustments were made in the cost figures. The latter therefore include the actual or estimated cost of mold loft work. No charges were made for calking performed after water was admitted to the tank.

¹ Weights include bounding bars, but do not include rivets used to attach these bars to the test tank.

² Weight was taken before bulkhead had been cut to receive pipes and channels.

CHAPTER IV

BEHAVIOR OF BULKHEADS UNDER TEST

An analysis of the deflection and strain-gage observations taken on all bulkheads is given farther on. A brief description of the behavior of the various bulkheads under test and of the modes of failure in each case is given here for the purpose of giving, first, a general idea of the results obtained.

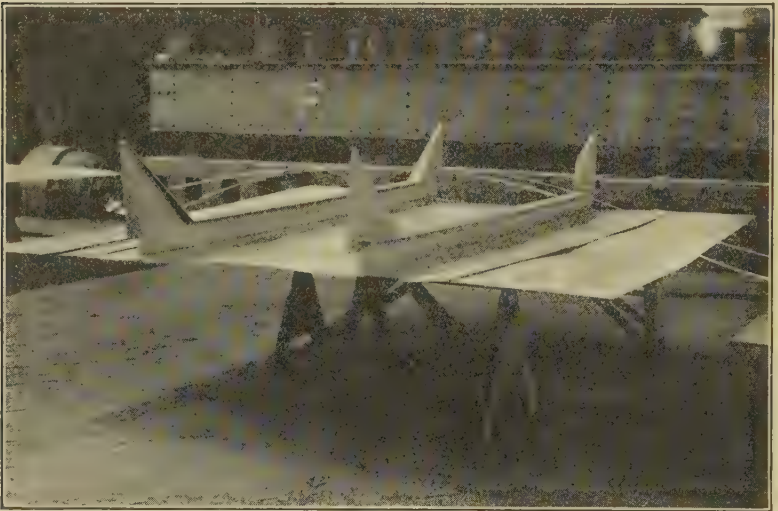


FIG. 20.—Type B welded bulkhead prior to installation.

The shop welding on this has been completed. Note the 10.2-lb. closer pieces, placed on the bulkhead.

Bulkhead R_1 (A 10.2-lb. Bulkhead Completely Riveted with Angle Bracket Clips Single Riveted to Tank).—The riveting on this bulkhead proved to be excellent as to workmanship, practically no leaks developing under test heads up to 50 ft. from the bottom of the tank. Even at this head the leaks through the face of the bulkhead were minor. At a 58-ft. head the extent of the leakage increased, but it was still not great except through the bracket connections to the top and bottom of the tank,

where streams of water poured from around the rivets. At a 66-ft. head the bulkhead was still intact and apparently able to withstand this pressure for an indefinite time. The deflection of the stiffeners at mid-length increased gradually up to a



FIG. 21.

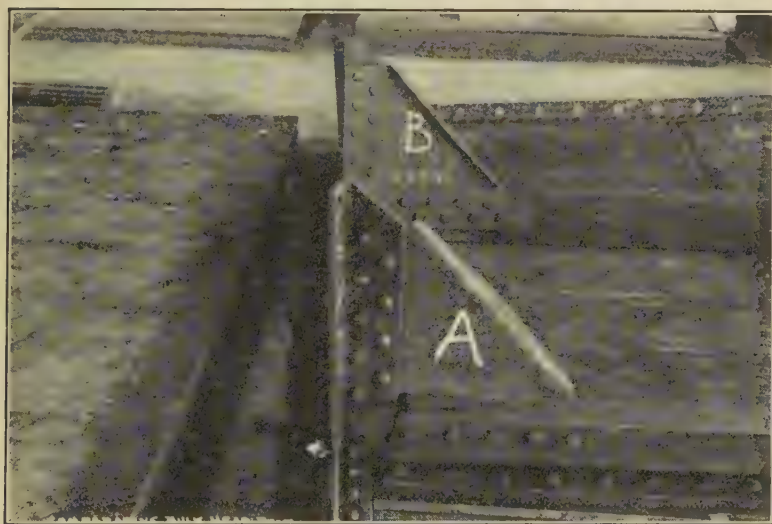


FIG. 22.

maximum of about $4\frac{1}{2}$ in. at the 66-ft. head. No deflection readings were taken on the bulkhead plating at this head, but at a 52-ft. head readings were taken which showed the plating had bulged out about 1 in. beyond the stiffeners. An exami-

nation of the bulkhead after the test showed that all riveting was intact except that through bracket clips. Rivets in these locations had been stretched considerably, as can be seen from Fig. 22. Both stiffeners failed by crumpling at about mid length, as shown in Fig. 21.

Bulkhead A (A 10.2-lb. Welded Bulkhead with Tee-bar Boundary Riveted to Tank, One Stiffener with Type 1 Brackets and One with Type 2 Brackets).—The plating of this bulkhead had been considerably and irregularly distorted during welding operations so that some parts of it had initial inward deflections while other parts were distorted outward. While the head on the bulkhead was being raised from 20 to 28 ft., heavy panting was noted in the plating between stiffeners. The period of these oscillations was estimated as 1 sec. and the amplitude about 1 in. At a 60-ft. head cracking noises were heard, and the stiffener with type 1 brackets suddenly deflected about twice the amount existing previously. At a 62-ft. head the other stiffener suddenly deflected considerably, with accompanying cracking noises. A small hole opened in the plating in way of the stiffener with type 1 brackets. This hole was about 6 in. from the top of the bulkhead and was caused evidently by the previous failure of the bracket. A stream of water from $\frac{1}{8}$ to $\frac{1}{4}$ in. diameter spurting from the hole. At the maximum head of 66 ft. another slight leak developed, this time in the welding to boundary bar at the top of the stiffener with type 1 brackets. Except for the two leaks mentioned, the bulkhead was practically drumtight throughout the test. The bulkhead appeared capable of resisting the maximum head applied for an indefinite period. The deflections and permanent sets of the stiffeners, particularly at moderate heads, were very much less than those of bulkhead R₁. The stiffener with type 1 brackets deflected more than that with type 2 brackets, and the failure of the former undoubtedly hastened that of the latter. Except for the panting described, the plating behaved in much the same manner as had that of bulkhead R₁.

An examination of the bulkhead made after the test showed that the stiffener with type 1 brackets had broken completely through at the top and bottom brackets and also had failed by crumpling at about mid-length. The stiffener with type 2 brackets tore away from the tank at the top by rupturing the tee weld, broke along the butt weld connecting the bottom

bracket to the web, tore across the face plate at the bottom bracket, and failed by crumpling at about mid-length. The damage sustained is shown in Figs. 23 and 24. Ruptured welds

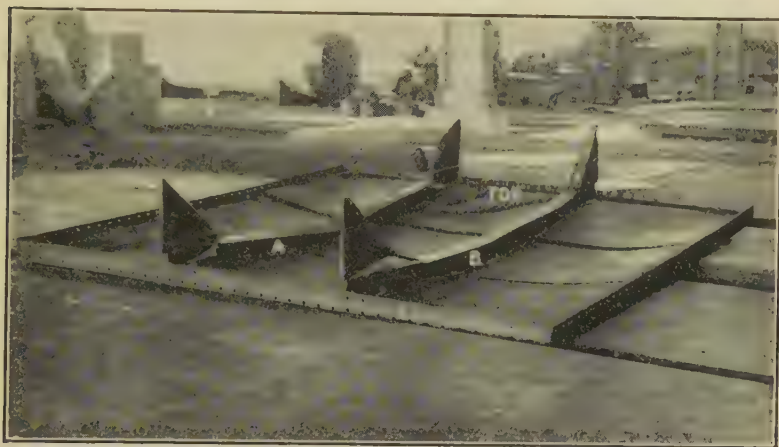


FIG. 23.—Type A welded bulkhead after removal from tank.

The damage to the stiffeners is clearly visible. Note the top and end of B stiffener where the bracket pulled away from the top of the tank.



FIG. 24.—End view of type A welded bulkhead showing distortion of stiffeners.

were found to be of good quality except the tee weld at top of the upper type 2 bracket, which showed evidences of insufficient penetration.

Bulkhead B (A 7.65-lb. Bulkhead Completely Welded, One Stiffener with Type 1 Brackets and One with Type 2 Brackets).

Except for the absence of panting of plating, the behavior of this bulkhead under test was generally similar to that of bulkhead A. At a 64-ft. head both type 1 brackets failed, and the stiffener deflection increased suddenly by several inches. A few moments later the bottom bracket of the other stiffener failed, then the top bracket. Simultaneously a crack about 12 in. long opened in the welded lap in way of the top type 1 bracket. The failures were accompanied by a series of cracking noises. The deflections of the stiffeners during the test were of the same order as those for bulkhead A, but the plating deflected somewhat more.

Examination of the bulkhead after the test showed that none of the brackets had pulled away from the tank. Figures 25,

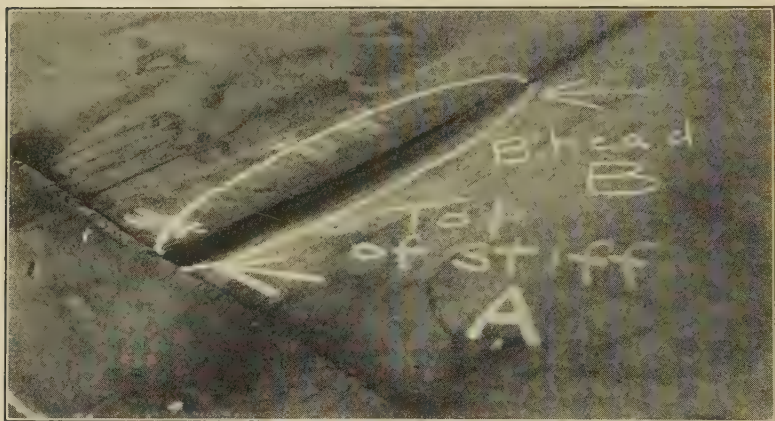


FIG. 25.—Split in welding at the top of stiffener A, bulkhead B.

This picture shows the face of the bulkhead, and the rear view of the same break is visible in Fig. 27. The ends of the break are indicated by the arrows.

26, and 27 show the damage sustained by this bulkhead except the buckling of stiffeners at about mid-length. Examination showed the fractured welds on this and subsequent bulkheads to be of good quality.

Bulkhead C (A 6-lb. Completely Welded Bulkhead, One Stiffener with Type 1 Brackets and One with Type 2 Brackets).—Except for slightly greater deflections and except for minor panting at low heads, this bulkhead behaved in much the same way as had bulkhead B. While the head was being increased from 56 to 58 ft. top and bottom type 1 brackets failed. A few moments later and at the same head the adjacent welded plate lap opened up for a distance of about 4 ft. in the outer weld and

5 ft. in the inner one. The resulting leakage was so great that the test was discontinued.

During the test the plating had a decided tendency to open up along the laps without rupture of the welds except in the

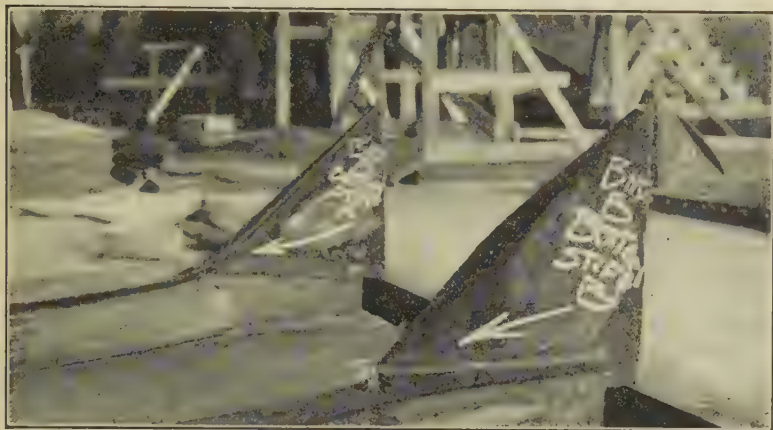


FIG. 26.—Damage to bottom of stiffeners of bulkhead B.

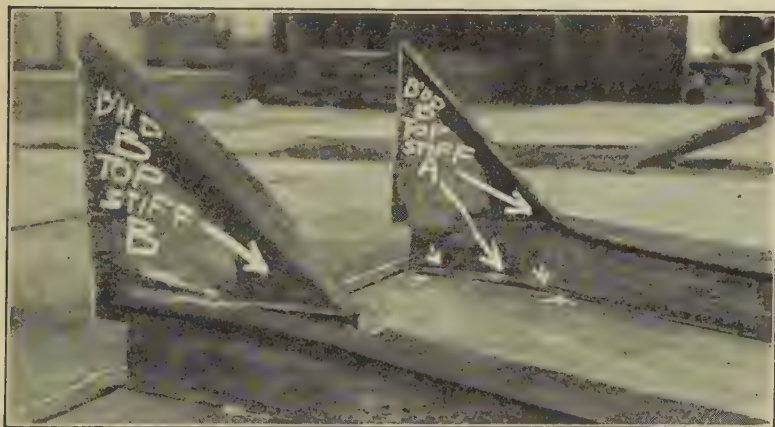


FIG. 27.—Damage to top of stiffeners of bulkhead B.

Note break indicated by arrows and referred to in description of Fig. 27.

instance cited. After the test the amounts by which the plating was separated along the laps were measured through a series of small holes drilled through one thickness of plating. The measurements taken showed that the two courses of plating forming the laps had separated by distances varying from $\frac{1}{4}$ to $1\frac{3}{8}$ in.

Figures 28, 29, and 30 illustrate the damage sustained by this bulkhead, except the buckling which occurred at mid-length of stiffeners.



FIG. 28.—Crack in the welded lap at the bottom of bulkhead C in way of B stiffener.

The total length of this crack was between 3 and 4 ft. Note the rule used for making deflection measurements; the point of this has been hardened to prevent wear.

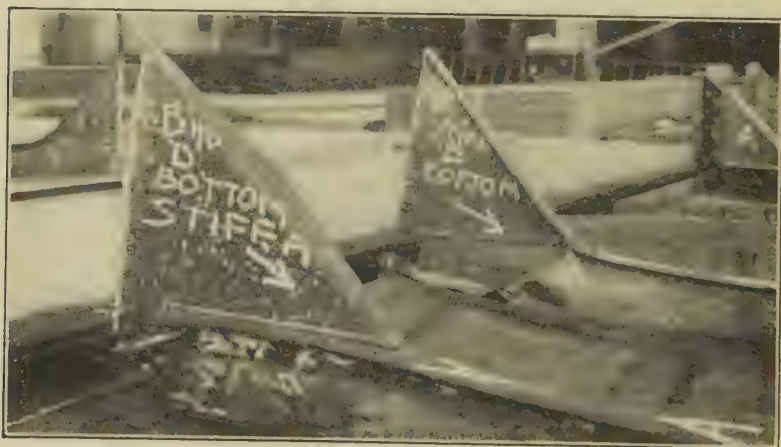


FIG. 29.—Damage to the bottom brackets of the stiffeners on bulkhead C. Note that B stiffener is intact, but that the plating has broken away along the welded lap.

Bulkhead D (A 6-lb. Completely Welded Bulkhead with Panels Pressed in Plating between Stiffeners, One Stiffener with

Type 1 Brackets and One with Type 2 Brackets).—The behavior of this bulkhead was a distinct disappointment. From the

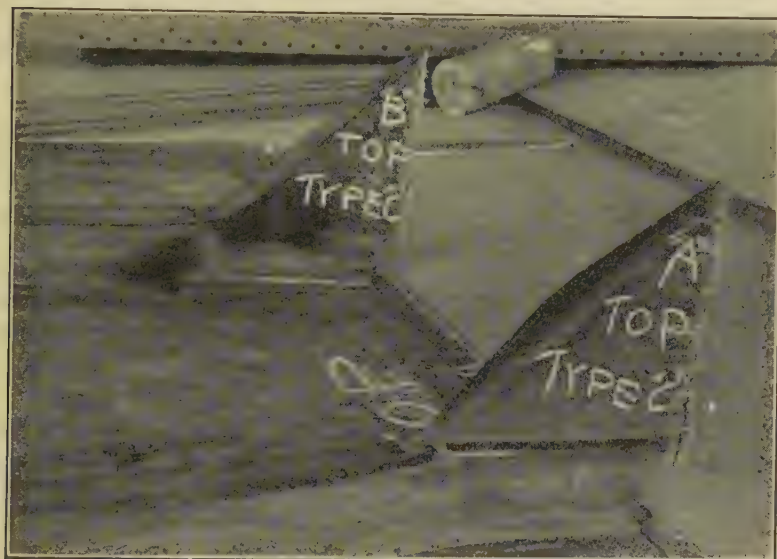


FIG. 30.—Damage to top of A stiffener, bulkhead C.
Note that stiffener B is intact.

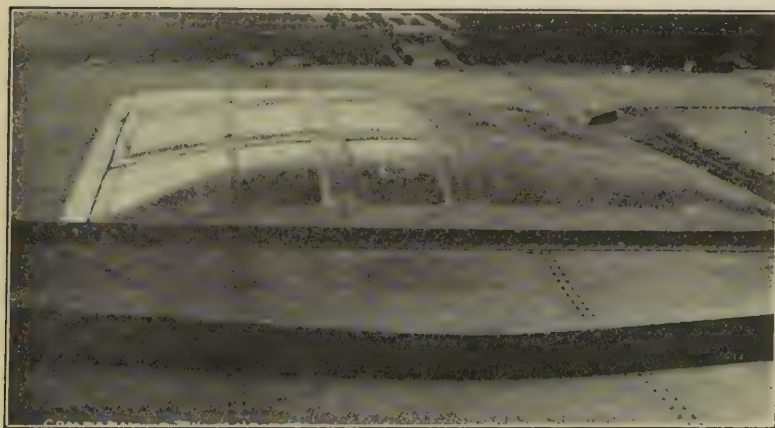


FIG. 31.—Front view of bulkhead D.

Note the way in which the panels have been pulled almost completely out and the fact that the plating in way of the stiffeners has deflected out almost as far as the remaining plating.

outset the deflections were more than those of any bulkhead previously tested. At a 48-ft. head both type 1 brackets failed,

and at a head between 50 and 52 ft. both type 2 brackets gave way. As the head was increased to a maximum of 66 ft. the deflection increased to a total of about 12 in., and the sides of



FIG. 32.—Damage to stiffeners at top of bulkhead D.

Both stiffeners broke completely through and the plating pulled entirely away from the bracket, leaving the bracket secured to the top of the tank.



FIG. 33.—Damage to bottom of stiffeners, bulkhead D.

Stiffener A parted in the weld and stiffener B behaved as had the top brackets of both stiffeners shown on Fig. 32.

the tank were bent in as much as 3 in. Although the bulkhead was perfectly tight at this head, it was decided to discontinue the test in order to avoid further damage to the tank.

The damage to brackets was found to be similar to that in previous cases. None of the brackets tore away from the tank. Figures 31, 32, and 33 give views of the ruptured brackets and of the face of the bulkhead.

Bulkhead E (A 10.2-lb. Welded Bulkhead with Welded Lug End Connections for Stiffeners and with Tee-bar Boundary

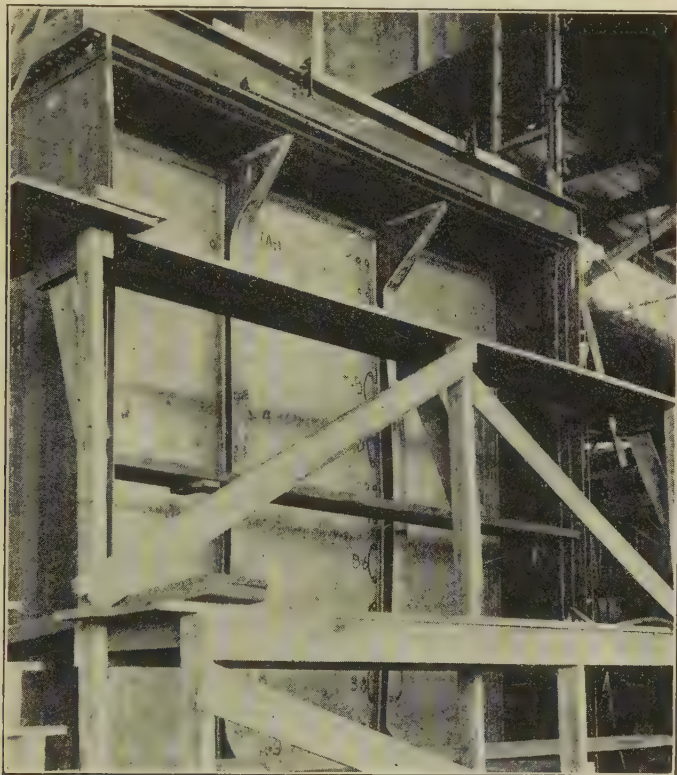


FIG. 34.—General view of type F bulkhead.

Showing distortion in stiffeners after being subjected to accidental head of 35 lb. per square inch. Note plank inside of tank resting on clamps attached to either side of the tank. This plank was in place during remainder of the test and served to indicate the absence of any further distortion of the sides of the tank.

Riveted to Tank).—This bulkhead deflected considerably more than any previously tested except bulkhead D. At a head of between 42 and 44 ft. both stiffeners pulled away from the top and bottom of the tank. The bulkhead was otherwise intact and free from leaks, but in order to avoid further deformation of the tank it was decided not to go to higher heads.

Bulkhead F (A 7.65 Completely Welded Bulkhead, One Stiffener with Type 2 Brackets and One with Type 3 Brackets).—This was the only bulkhead tested with stiffeners on the outside of the tank. While preparations were being made to put the bulkhead under test an unfortunate accident occurred which destroyed the usefulness of strain and deflection data taken



FIG. 35.—General view of bulkhead F after completion of tests.

Note patch welded over break in weld at upper right-hand corner of bulkhead and break through welding at lower right-hand corner. Note also extreme deformation of both stiffeners.

subsequently on this bulkhead. With the filling valve to the tank open and that to standpipe closed, the vent was inadvertently closed, with the result that the bulkhead was suddenly subjected to the full main pressure (read by gage as 35 lb. per square inch). This resulted in considerable deflection and permanent set in the stiffeners and plating and also caused a leak through the welded joint in the upper right-hand corner

of the bulkhead. The pressure was relieved, and the broken weld was cut out and rewelded. Pressure heads up to a maximum of 66 ft. were then applied. At this head the stiffeners were badly distorted, as can be seen from Fig. 35, but the brackets and their connections to the tank were not ruptured. Two leaks, one at the upper and one at the lower right-hand corner, prevented the test from being carried to greater heads.



FIG. 36.—Type G bulkhead after withstanding head of 64 ft.
Note the pipes and channels piercing the bulkhead.

Bulkhead G (A 7.65-lb. Completely Welded Bulkhead, One Stiffener with Type 4 Brackets and One with Type 5 Brackets).—This bulkhead was pierced by four sections of channels and by four sections of plugged pipe. At a head between 56 and 58 ft. the welding opened up at the corner of one of the 10-in. channels. Although there was considerable leakage through this crack the head was carried up to 64 ft. Examination showed

TABLE 3.—PERFORMANCE OF BULKHEADS UNDER HYDROSTATIC TEST

Bulkhead	Description	Head at failure	Maximum head	Mode of failure	Average deflection at mid-length stiffeners 36 ft. head	Average permanent set at mid-length stiffeners after 36 ft. head	Maximum total deflection of plating at mid-height of bulkhead, 36 ft. head	Remarks
R. Fig. 9	10.2-lb. completely riveted; single angle bracket clips	66 ft. (about)	66 ft.	Crumpling of stiffeners pulling away of ends of bracket clips	1.3 in.	0.6 in.	1.9 in.	Considerable leakage at heads above 58 ft.; bracket clips pulled away from tank slightly
A Fig. 10	10.2-lb. completely welded; one stiffener with type 1 brackets, one with type 2 brackets	60–62 ft.	66 ft.	Rupture of type 1 brackets, followed by pulling away of one type 2 bracket and rupture of the other and by distortion of stiffeners at mid-length	0.5 in.	0.1 in.	1.7 in.	Bulkhead tight except for two small leaks after failure of brackets
B Fig. 11	7.65-lb. completely welded; one stiffener with type 1 brackets, one with type 2 brackets	64 ft.	64 ft.	Failure of type 1 brackets, followed by that of type 2 brackets and by distorting of stiffeners at mid-length	0.5 in.	0.1 in.	2.7 in.	Simultaneously crack about 12 in. long opened in plate lap
C Fig. 12	6-lb. completely welded; one stiffener with type 1 brackets, one with type 2 brackets	56–58 ft.	58 ft.	Failure of type 1 brackets, followed by opening up of 4–5 ft. of plate lap and by distortion of stiffeners at mid-length	0.8 in.	0.2 in.	3.3 in.	Bulkhead tight to failure when large leak occurred through plate lap

D Fig. 13	6-lb. completely welded; plating pressed into panels; one stiffener with type 1 brackets, one with type 2 brackets	Vertical stiffeners; vertical strakes of plating	48-52 ft.	66 ft.	Failure of type 1 brackets, followed by that of type 2 brackets and by distortion of stiffeners at mid-length	1.4 in.	0.8 in.	4.1 in.	Bulkhead tight at 66 ft., but deflected out about 1.2 in.
E Fig. 14	10.2-lb. completely welded; no brackets; both stiffeners with welded lug end connections	Vertical stiffeners; horizontal strakes of plating	42-44 ft.	44 ft.	All stiffener end connections pulled away from tank	2.5 in.	1.3 in. at 34 ft. head	3.4 in.	Bulkhead capable of taking greater pressure; bulkhead tight
F Fig. 15	7.65-lb. completely welded; one stiffener with type 2 brackets, one with type 3 brackets	Vertical stiffeners; vertical strakes of plating	66 ft. (about)	66 ft.	Stiffeners distorted by initial pressure accidentally applied; brackets did not fail	Not measured	Not measured	Not measured	Bulkhead subjected to initial suddenly applied pressure of 35 lb. per square inch; bulkhead tight at 66 ft. head except for two leaks in welds
G Fig. 16	7.65-lb. completely welded; one stiffener with type 4 brackets, one with type 5 brackets; four channel sections and four pipe sections pierced bulkhead	Vertical stiffeners; horizontal strakes of plating	Did not fail	64 ft.	No failure	0.5 in.	0.1 in.	1.3 in.	Only casualty was crack in weld at corner of 10 in. channel piercing bulkhead; bulkhead otherwise tight
R ₂ Fig. 9	10.2-lb. completely riveted; tee-bracket clips	Vertical stiffeners; horizontal strakes of plating	Did not fail	66 ft.	No failure	0.8 in.	0.3 in.	1.7 in.	Slight leakage above 40 ft. head; at 66 ft. head stiffeners "laid over," slightly; no pulling away of bracket clips

the stiffeners to be intact except for a slight permanent set. Brackets were undamaged and plating was tight except for the leak at the 10-in. channel. Figure 36 gives a view of the bulkhead after it had withstood the 64-ft. head.

Bulkhead R₂ (A 10.2-lb. Completely Riveted Bulkhead with Tee-bar Bracket Clips Double Riveted to Tank).—The double-riveted tee-bar bracket clips proved to be much more effective than the single-riveted angle clips of bulkhead R₁. The deflections and permanent sets of stiffeners on R₂ were considerably less throughout the test than had been the case with R₁. Bulkhead R₂ was tight up to a head of 40-ft., when slight weeps developed through the lower plate lap. At heads above 46 ft. some leakage occurred through the tee-bar bracket clip at the top of stiffener A, but this leakage was moderate even at the maximum head of 66 ft. Aside from the deflection of about 3½ in. there was no distortion of stiffeners at this head except a slight "laying over" sidewise. Bracket clips showed no signs of pulling away from the tank. Apparently the bulkhead was capable of withstanding the maximum head for an indefinite time.

In Table 3 the foregoing description of the behavior and failure of the various bulkheads has been put into convenient form for reference.

SPECIAL SHOCK TESTS

All heads greater than 16 ft. were applied to bulkheads F and G by means of a reciprocating pump discharging into the tank. No detrimental effects on the welds were noted.

After the failure of the stiffener end connections of bulkhead E, it was decided to conduct some special shock tests on this bulkhead. A pressure head of 36 ft. was placed on the bulkhead. A man then struck repeated blows on the face of the bulkhead with a heavy maul swung with all his strength. The blows were struck along the upper edge of the tank, at the corners, and down the side, as well as directly on the welded plate laps. The blows caused no cracks or leakage in any of the welds.

An additional welded 7.65-lb. bulkhead with tubular brackets was tested, and the brackets were found unsatisfactory. The bulkhead was then given a shock test consisting of tripping the latch on a manhole in the top of the tank when the bulkhead was under pressure. This was done several times at heads of 36 to 48 ft., and also when the tank was filled with water to

within 6 to 12 in. of the top, air pressure of 11 to 13 lb. per square inch being applied to the space over the water. In each case the sudden release of pressure caused the bulkhead to spring inward violently, but there was no apparent effect on any of the welding.

To throw further light on the effect of shock as well as to investigate means of eliminating warping in extremely light material, a three-sided welded structure was built of 4-lb. plate stiffened by 10.2-lb. flat bars. This structure, weighing about 900 lb., was dropped from a height of 18 ft. on a large plate of steel $\frac{3}{4}$ in. thick. Although considerable distortion occurred, none of the welds cracked in the slightest degree. The section was then dropped from a height of 30 ft. and landed directly on one of its corners. The damage to the welding was slight and local, extending 6 to 8 in. from the corner along two of the sides.

CHAPTER V

ANALYSIS OF DATA

Deflections.—In Figs. 37 and 38 there are shown plots of total deflections along a horizontal line 6 ft. from the bottom of the tank (line *D* of Fig. 8). The plots show the observed deflections connected by straight lines and are of interest chiefly in that they give a means of readily comparing the degrees of flexibility of the various designs of bulkheads. For instance, a marked superiority of R_2 over R_1 is evident, which can be attributed only to the improved bracket clips used in the construction of the former. It is also evident that there has been a decided gain in degree of rigidity in *G* as compared to *B*, both being 7.65-lb. bulkheads. In this case the improvement is due to the better design of brackets used on *G*. A comparison of *G* and R_2 is of special interest, for we see that the maximum deflection of the plating as well as that of the stiffeners is less for the 7.65-lb. welded bulkhead than for the better of the 10.2-lb. riveted ones.

Figure 39 shows the average deflections and permanent sets of the two stiffeners of each bulkhead, except *F*, which was omitted because of the injury done by the initial accidentally applied pressure. If the curves for *E*, which had no brackets, and *D*, the plating of which was pressed into panels, be disregarded, the superiority in degree of rigidity of the welded stiffeners over the riveted ones is quite apparent. This superiority exists in spite of the lower moments of inertia of stiffener cross-section existing on all welded bulkheads except *E* (see Table 4).

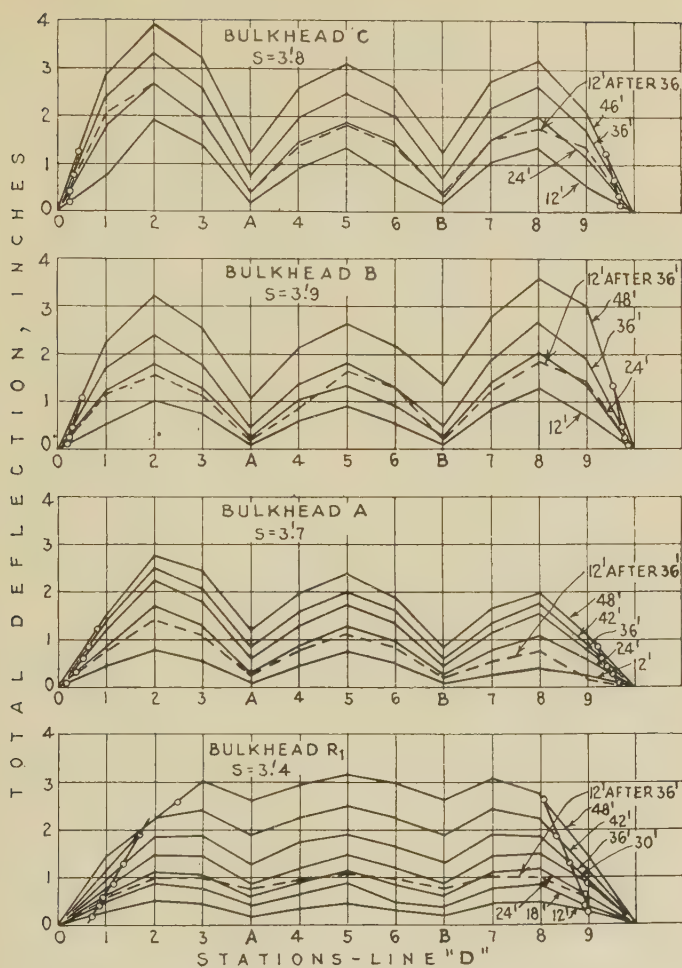


FIG. 37.—Deflections along horizontal line at mid-height of bulkheads R₁, A, B, and C for various heads from bottom of test tank.

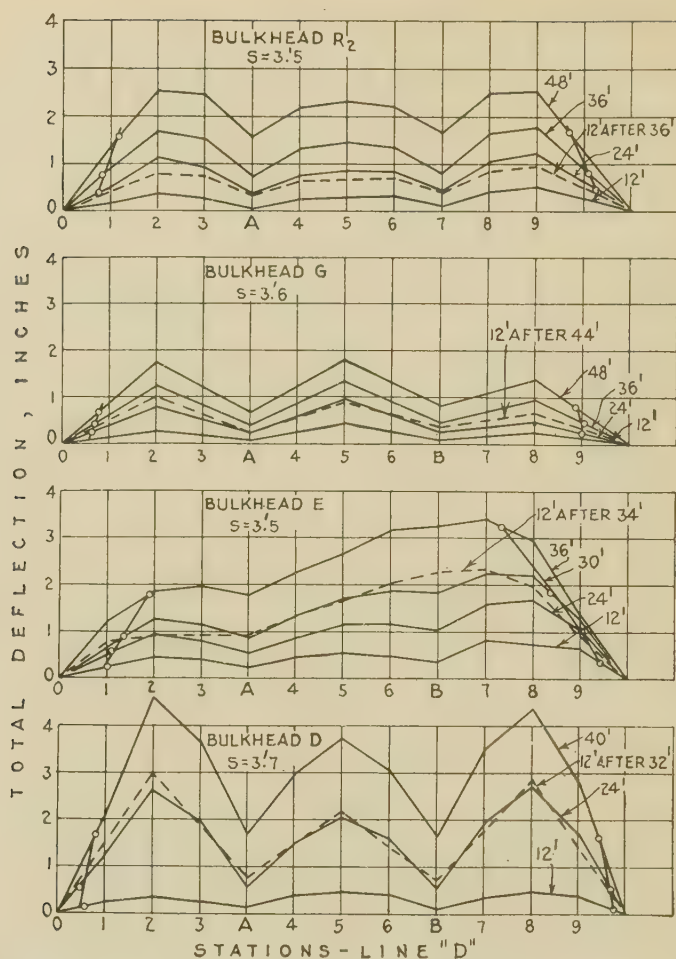


FIG. 38.—Deflections along horizontal line at mid-height of bulkheads D, E, G, and R₂ for various heads from bottom of test tank.

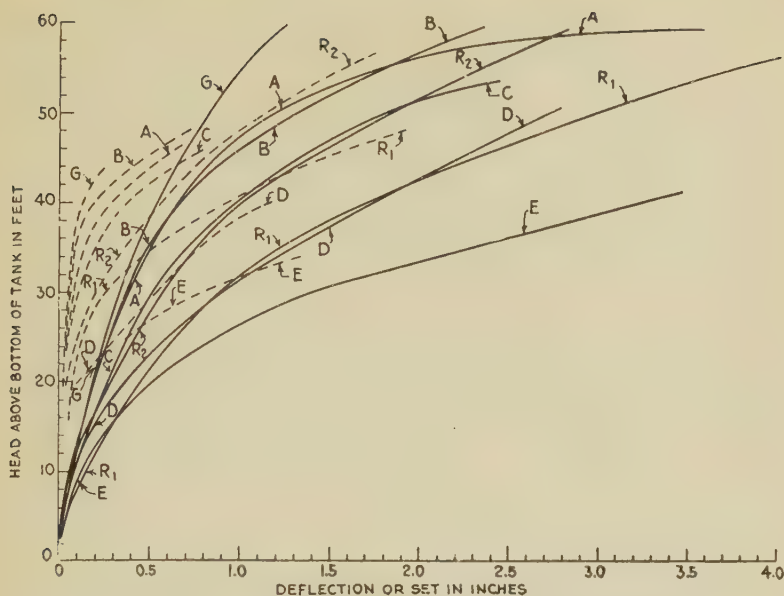


FIG. 39.—Curves of total deflection and permanent set.

Solid curves show average total deflections of mid-points of stiffeners. Dotted curves show average permanent set of mid-points of stiffeners. Letters on curves indicate type of bulkhead. Data averaged from two stiffeners.

TABLE 4.—MOMENTS OF INERTIA OF STIFFENERS ABOUT NEUTRAL AXES

Bulkhead	I-in. ⁴	I-in. ⁴
	30 tons, acting with stiffeners	60 tons, acting with stiffeners
R ₁ and R ₂	38.2	45.4
A.....	34.0	44.0
B and F.....	33.8	41.2
C.....	34.5	38.7
D.....	34.5	
E (no brackets).....	51.7	62.1
G.....	25.8	35.3

NOTE.—Rivet holes and areas of welds were disregarded; moments of inertia for bulkhead E are average of those at mid-height of the two stiffeners. Assumption on which columns 2 and 3 are based is that strips of plating 30 and 60 times, respectively, the thickness of bulkhead plating act with stiffeners.

Mathematical Treatment of Deflection and Strain Data.—A bracketed stiffener of a bulkhead under hydrostatic pressure may be considered as a beam partially or wholly fixed at the ends and carrying a uniformly increasing load from the top to the bottom of the stiffener. In Appendix C, it is shown, however, that without appreciable error the load may be taken as uniform. Under such an assumption the calculated end-bending moment corresponds to the average of the two end-bending moments for a similar beam having a uniformly increasing load.

The assumption will be made here, therefore, that a stiffener acts as a beam partially but equally fixed at the ends and carrying a uniform load corresponding to the head of water above mid-height of the bulkhead. This condition of loading is shown in Fig. 6, the following notation applying to this figure and to subsequent mathematical analysis:

- l = length of stiffener, in inches
- h = head of water, in feet, above mid-height of bulkhead
- s = width, in feet, of bulkhead load borne by each stiffener
- w = hydrostatic load per inch of length of stiffener for uniform head of 1 ft.
- M = bending moment at any point, in inch-pounds
- δ = deflection at any point, in inches
- E = Modulus of elasticity in pounds per square inch = 30,000,000
- I = Moment of inertia of cross-section of stiffener about neutral axis, in in.⁴. A strip of plating of unknown width is assumed to act with each stiffener
- t = thickness of bulkhead plating, in inches
- f = tensile stress, in pounds per square inch, in face of bulkhead plating
- y = distance, in inches, from neutral axis to face of bulkhead plating
- f' = tensile stress, in pounds per square inch, in extreme fiber of stiffener flange or face plate
- y' = distance, in inches, from neutral axis to outside of stiffener flange or face plate
- k = fixity factor, or ratio between end-bending moment for a stiffener and the end-bending moment if ends were fixed
 - subscript "max" signified maximum value
 - subscript "O" signifies value at origin, or top of stiffener
- x = distance from origin of any point, in inches
- x_1 = distance, in inches, from origin to upper point of zero bending moment
- x_2 = distance, in inches, from origin to lower point of zero bending moment

From the fundamental differential equation of the theory of the elastic deflection of beams, it is possible to deduce the following expressions:

$$M = whl \frac{x}{2} - \frac{whx^2}{2} - k \frac{whl^2}{12} \quad [1]$$

$$EI\delta = \frac{whlx^3}{12} - \frac{whx^4}{24} - \frac{kwhl^2x^2}{24} + (k-1) \frac{whl^3x}{24} \quad [2]$$

$$\delta_{\max} = -\frac{whl^4}{384EI}(5-4k) \quad [3]$$

$$k = \frac{6}{l^2}x_1x_2 \quad [4]$$

We also have from the beam theory the following well-known equations:

$$f = \frac{My}{I} \quad [5]$$

$$f' = \frac{My'}{I} \quad [6]$$

To bring as much as possible of the observed deflection and terrain data into use in determining the unknowns of these expressions, these data were plotted on a non-dimensional basis (Figs. 40 to 54).

For nine equally spaced stations on each stiffener, the deflection curves show the rate of change of elastic deflection (total deflection less permanent set) with change in head up to the point where this rate departs from a constant value. The elastic deflection for any point at a given head (not below the top of the tank) is obtainable by multiplying the abscissa for that point by the head above mid-height of the bulkhead. Curves of permanent set were included in Figs. 40 to 47 to supplement the data of Fig. 39.

As somewhat erratic strain-gage readings had been obtained, it was considered desirable to indicate on Figs. 48 to 54 the mean of the readings for the various stations along the stiffeners divided by the corresponding heads above mid-height of the bulkheads. In general, observed points were clustered closely (within ± 100) along the dotted curves of these figures, but there were a number of widely scattered points.

The smooth curves of Figs. 48 to 54 represent the best faired average of the strain observations for each pair of stiffeners. For a given head (not below the top of the bulkhead) the average plate stresses in way of stiffeners can be obtained by multiplying

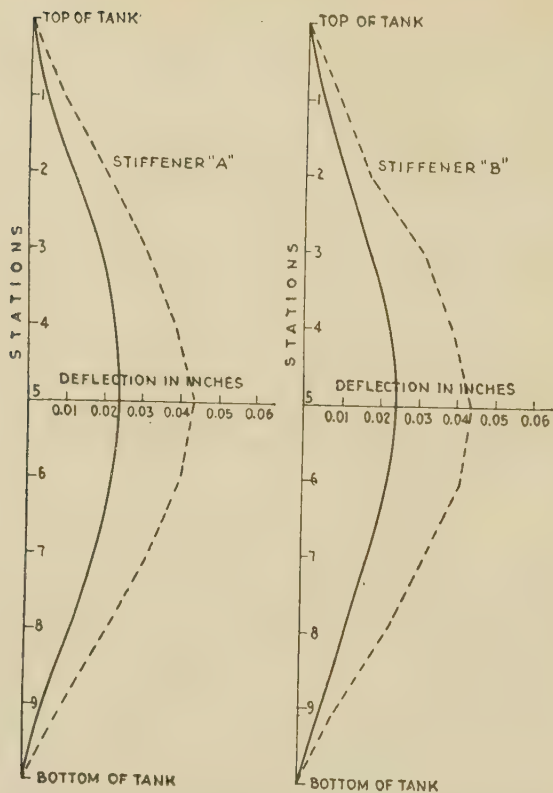


FIG. 40.—Riveted bulkhead type R_1 deflection curves.

Solid curves show average elastic deflections per foot of head. Dotted curves show total deflection at 36 ft. head above bottom divided by head above middle of tank.

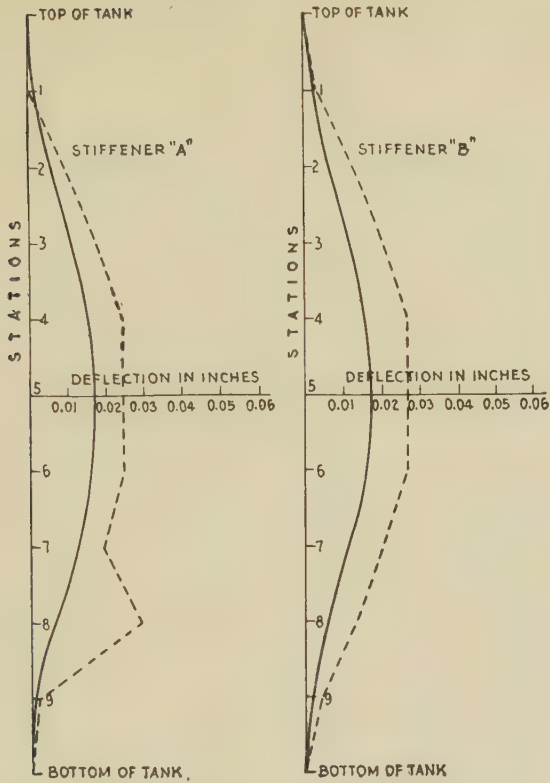


FIG. 41.—Riveted bulkhead type R_2 deflection curves.

Solid curves show average elastic deflections. Dotted curves show total deflection at 36 ft. head above bottom divided by head above middle of tank.

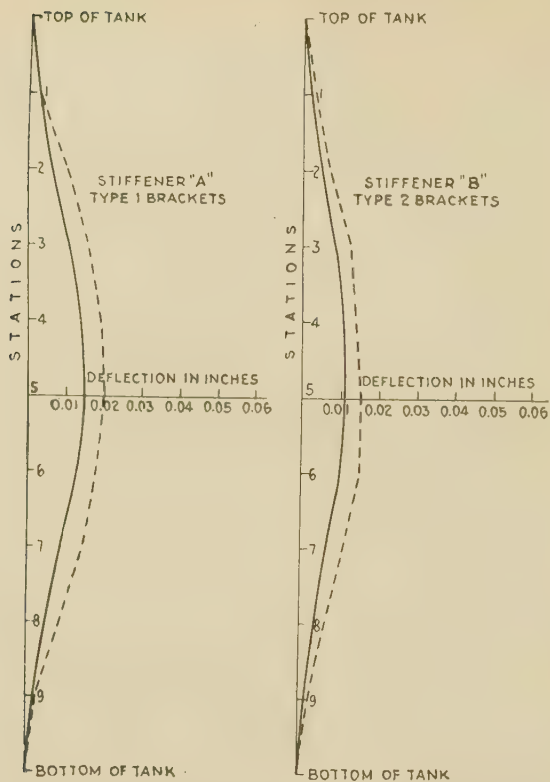


FIG. 42.—Welded bulkhead type A deflection curves.

Solid curves show average elastic deflections per foot of head. Dotted curves show total deflection at 36 ft. head above bottom divided by head above middle of tank.

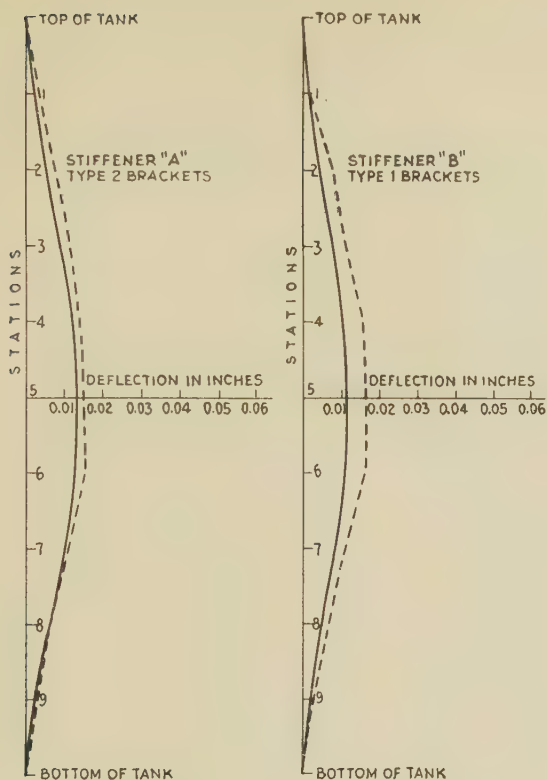


FIG. 43.—Welded bulkhead type B deflection curves.

Solid curves show average elastic deflections per foot of head. Dotted curves show total deflection at 36 ft. head above bottom divided by head above middle of tank.

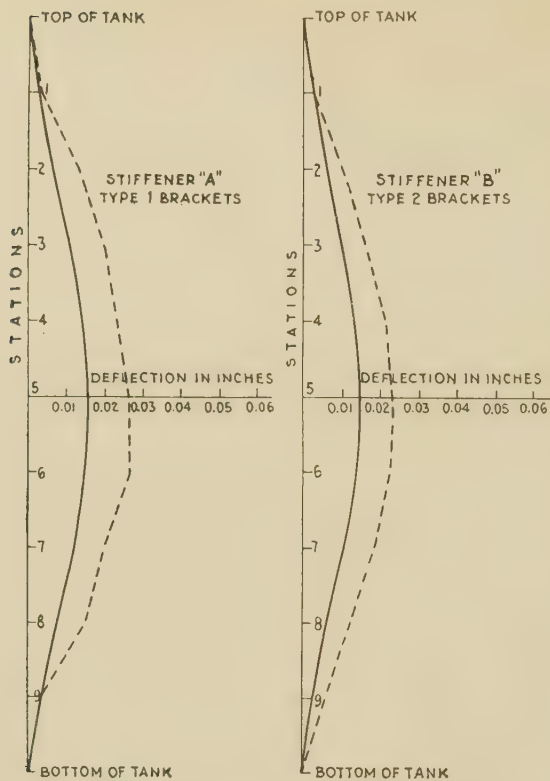


FIG. 44.—Welded bulkhead type C deflection curves.

Solid curves show average elastic deflections per foot of head. Dotted curves show total deflection at 36 ft. head above bottom divided by head above middle of tank.

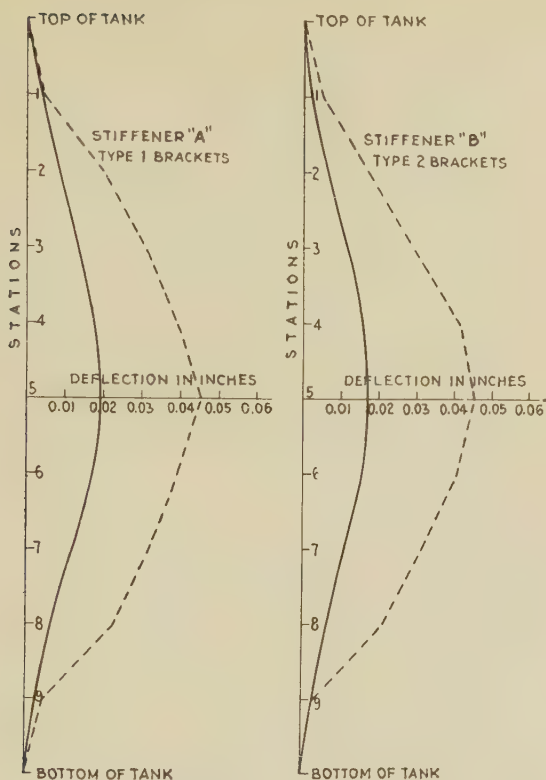


FIG. 45.—Welded bulkhead type D deflection curves.

Solid curves show average elastic deflections per foot of head. Dotted curves show total deflection at 36 ft. head above bottom divided by head above middle of tank.

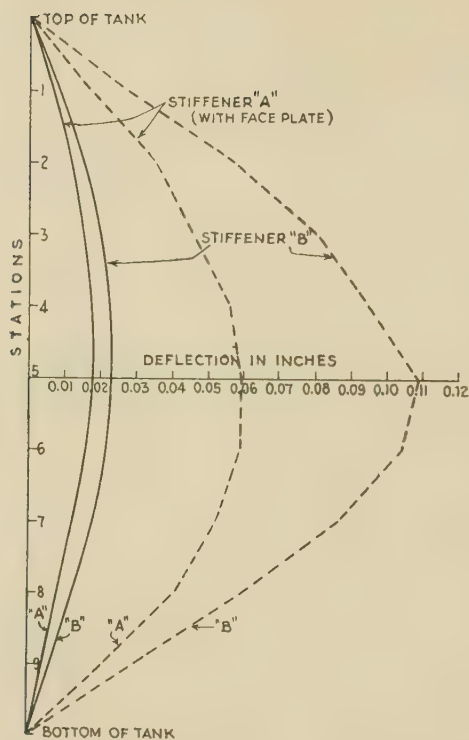


FIG. 46.—Welded bulkhead type E deflection curves.

Solid curves show average elastic deflections per foot of head. Dotted curves show total deflection at 36 ft. head above bottom divided by head above middle of tank.

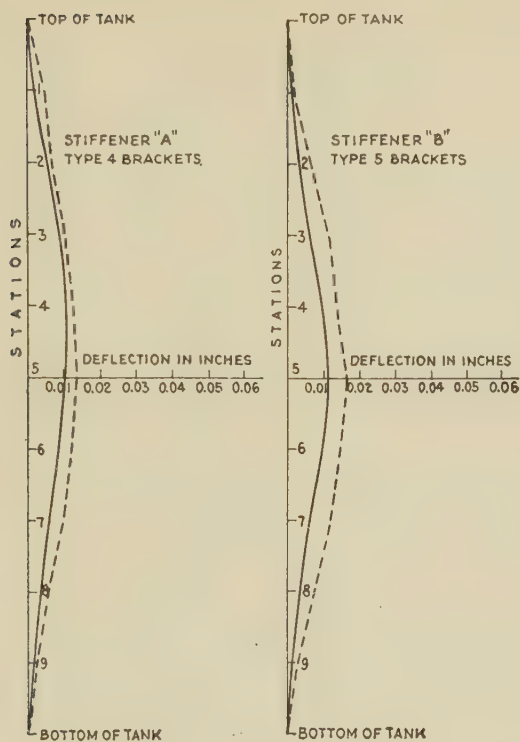


FIG. 47.—Welded bulkhead type G deflection curves.

Solid curves show average elastic deflections per foot of head. Dotted curves show total deflection at 36 ft. head above bottom divided by head above middle of tank.

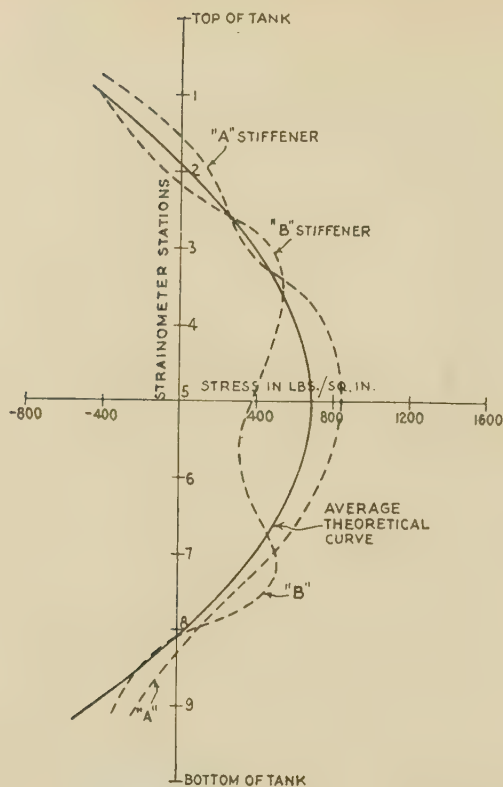


FIG. 48.—Riveted bulkhead type R₁.
Curves of stress per foot of head based on strainometer readings.

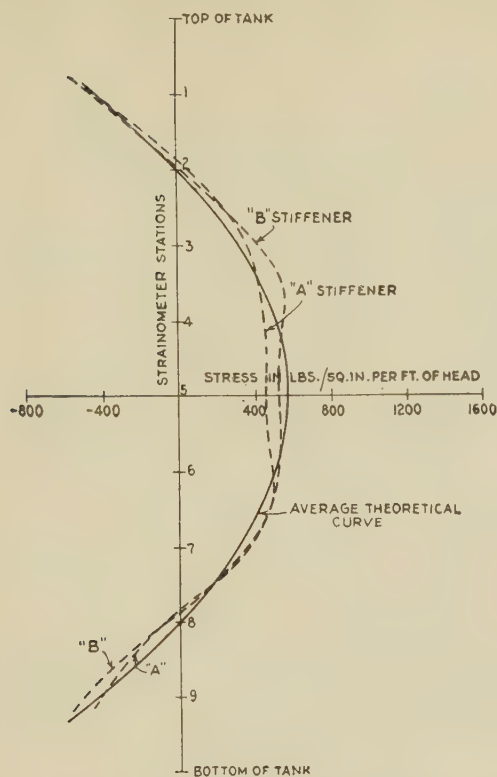


FIG. 49.—Riveted bulkhead type R₂.

Curves of stress per foot of head based on strainometer readings. Heads are measured from middle of tank.

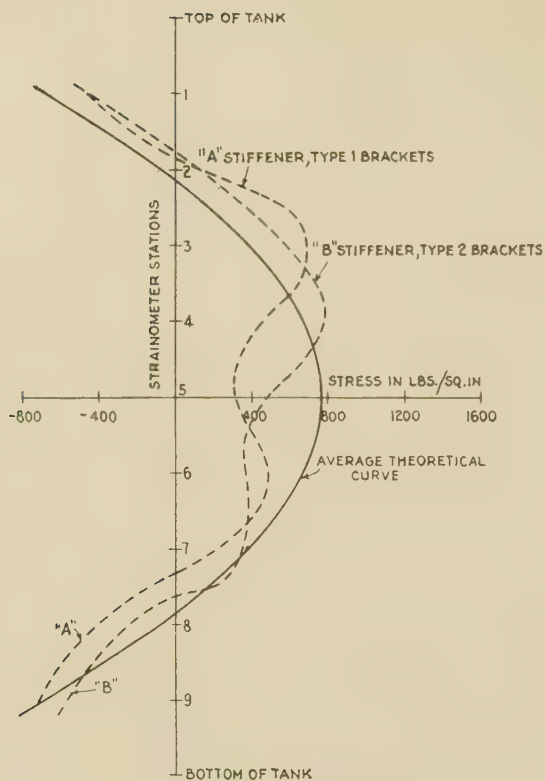


FIG. 50.—Welded bulkhead type A.
Curves of stress per foot of head based on strainometer readings.

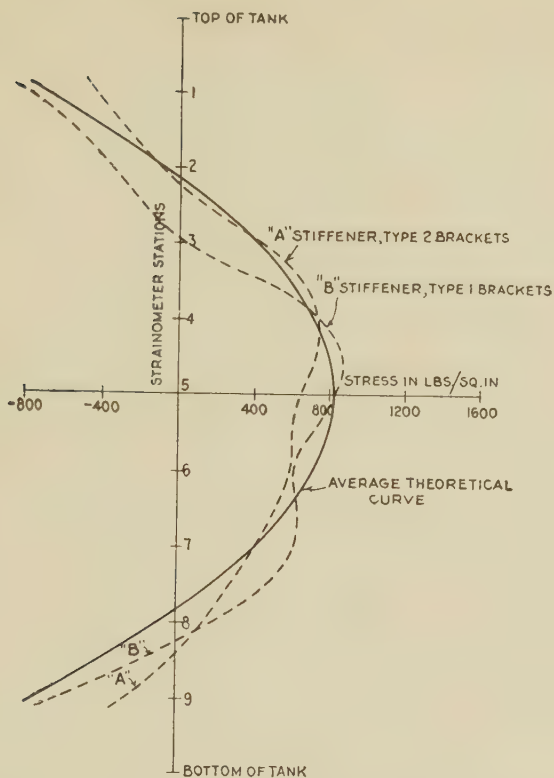


FIG. 51.—Welded bulkhead type B.
Curves of stress per foot of head based on strainometer readings.

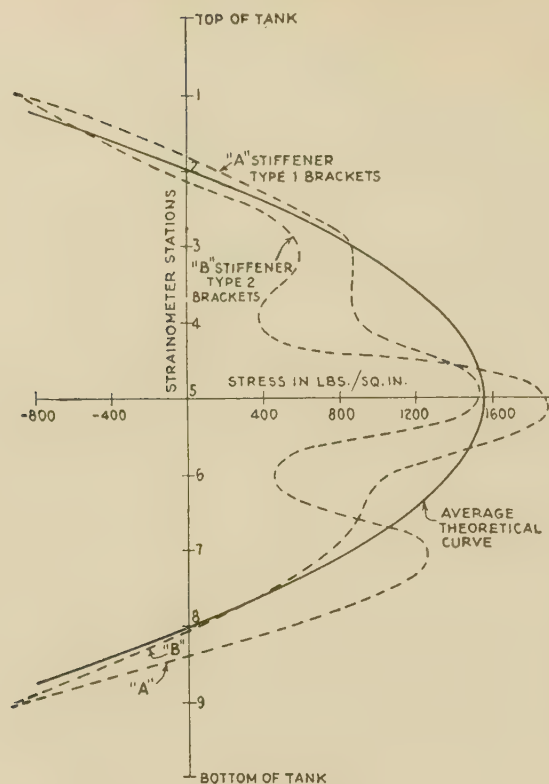


FIG. 52.—Welded bulkhead type C.
Curves of stress per foot of head based on strainometer readings.

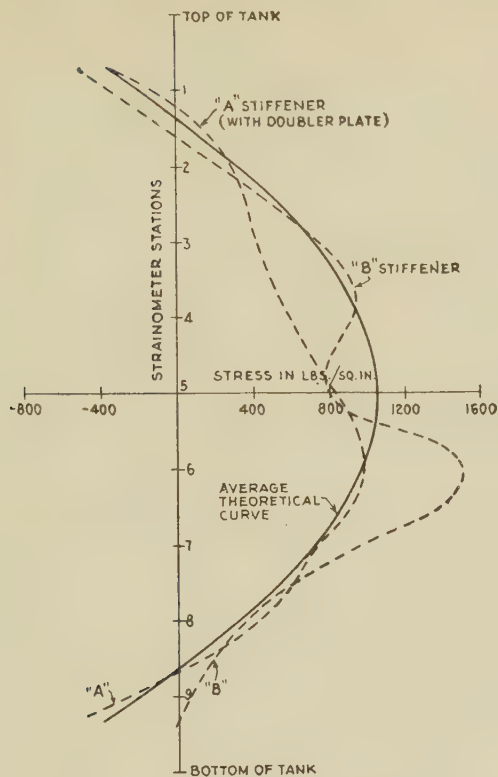


FIG. 53.—Welded bulkhead type E.
Curves of stress per foot of head based on strainometer readings.

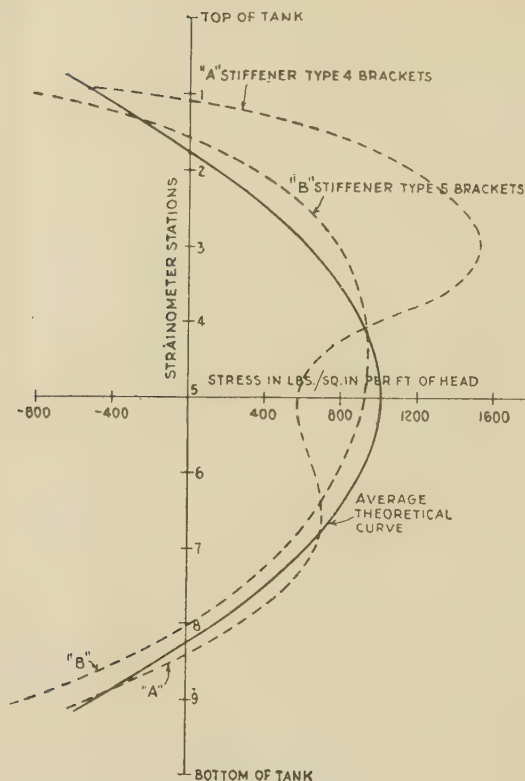


FIG. 54.—Welded bulkhead type G.
Curves of stress per foot of head based on strainometer readings. Heads are measured from middle of tank.

the abscissas of the smooth curve for the bulkhead concerned by the head above mid-height of the bulkhead. Obviously the curves are not applicable beyond the elastic limit of the material of the stiffeners. Plots have not been included for bulkhead D as its performance was extremely erratic and unsatisfactory.

The smooth curves of Figs. 48 to 54 give us the values of $\frac{f_{\max}}{h}$, x_1 , and x_2 , while the deflection curves of Figs. 40 to 47 give us the corresponding values of $\frac{\delta_{\max}}{h}$.

To ascertain the width of stiffener load s , use was made of the plots of Figs. 37 and 38. For each head the point on the plating between stiffener and tank side, and having the same deflection as the stiffener, was marked. A pair of curves for each bulkhead were drawn through the points so located. The effective width of the bulkhead for any head was assumed to be the distance at that head between the pair of curves. For each bulkhead the width of stiffener load s was computed from the average effective width of bulkhead determined on the foregoing assumption. The values of s are given in Figs. 37 and 38.

By making use of equations [1] to [6], we are now in a position to proceed with the determination of the values of k , I and I/y for all bulkheads tested except D and F. We may also calculate the corresponding values of y , y' and f' and the widths of bulkhead plating acting with the stiffeners. Table 5 gives the results of these calculations.

The following points of interest will be noted from Table 5:

(1) The end fixity factor is almost unity for all bracketed, welded bulkheads, except G, and the low factor calculated for this bulkhead appears to be in error. The factor for the welded bulkhead with lug end connections, E, is 0.72, while those for the riveted bulkheads are 0.92 and 0.96.

(2) For all bracketed, welded bulkheads the values of I calculated from the deflection observations are considerably higher than those based on the strain data. These discrepancies may be partially or wholly accounted for by the influence of the brackets in raising the average I of the stiffeners. It is this average I which fixes the deflection, whereas the stress at any point is determined by the moment of inertia at the section where the stress is measured.

TABLE 5.—RESULTS OF CALCULATIONS FROM OBSERVED DEFLECTION AND STRAIN DATA

Bulkhead	(1) k	(2) s	At mid-height		(5) Weight water per cw. ft.	(6) w	(7) $E/10^6$	(8) I Calcu- lated from deflection observa- tions	Calculated from strain observations					(15) Head above bottom at which f' was 42,000 lb. per square inch	
			(3) δ/h	(4) f/h					(9) I/y	(10) y	(11) I	(12) y'	(13) f'/h		(14) Width of plating acting with stiffener
R_1	0.92	3.4 ft.	0.023	680	61.2	17.3	30	37.1	25.5	1.78	45.3	4.47	1,710	65 t	31 ft.
A	0.99	3.7 ft.	0.013	770	61.2	18.9	30	56.5	21.6	2.10	45.4	4.15	1,520	62 t	34 ft.
B	0.99	3.9 ft.	0.012	820	61.2	19.9	30	64.3	21.4	2.13	45.6	4.25	1,640	90 t	32 ft.
C	0.96	3.8 ft.	0.015	1,550 ²	61.2	19.4	30	56.0	11.7	2.94	34.5	3.36	1,770	30 t	30 ft.
E	0.72	3.5 ft.	0.020	1,050	61.2	17.8	30	70.3	22.8	2.57	58.6	4.06	1,660	53 t	31 ft.
G	0.88 ¹	3.6 ft.	0.011	1,010	61.2	18.3	30	64.4	15.9	2.43	38.6	3.76	1,560	77 t	33 ft.
R_2	0.96	3.5 ft.	0.017	570	61.2	17.8	30	45.3	29.1	1.63	47.4	4.62	1,610	77 t	32 ft.

¹ This appears to be in error as bulkhead G was much stiffer than bulkhead B, which had the same weight of plating. Calculations for bulkhead G in later columns were made on the assumption that the correct value of k was .99.

² This figure is probably about 25 per cent too high because of the opening up of plate lap, throwing strain gage points further away from the neutral axis.

(3) Except for bulkhead C, the strain readings of which appear to be in error, the calculated widths of bulkhead plating acting with stiffeners, column (14), range from fifty-three to ninety times the thickness of plating, a conservative average value being sixty times the thickness.

(4) The calculated heads at which the yield point of the stiffener material is reached, as given in column (15), are in fair agreement with the curves of permanent set of Fig. 39.

The calculated stresses of column (13), had they been known before the test, would have led one to expect much earlier failure for all bulkheads than was actually the case. The great resistance of the bulkheads beyond the elastic limit was apparently owing to the increasing support received from the sides and ends of the tank, as the bulkhead plating became more and more deflected. Bulkhead D was an extreme example of this action. In this case the 6-lb. plating withstood a head of 66 ft. practically unassisted by the stiffeners, the deflection under this head being about 1 ft.

For bulkheads of great width, the following relations for the stiffeners not adjacent to bounding bulkheads may be deduced from the mathematical equations given in the paper:

$$M\alpha hl^2$$

$$f'_{\max}\alpha \frac{hl^2y}{I}\alpha \frac{hl^3}{I}$$

(assuming y to vary with the length of the stiffeners)

Since $f'_{\max}$ is a constant determined by the design,

$$I\alpha hl^3$$

$$\delta_{\max}\alpha \frac{hl^4}{I}\alpha \frac{hl^4}{hl^3}$$

$$\delta_{\max}\alpha^l$$

We see from the foregoing that the maximum deflection may be expected to vary as the length of the stiffeners (or the height of the bulkhead). On the other hand, the plating between stiffeners will deflect beyond the stiffeners by an amount dependent on the head rather than the height of bulkhead. The total deflection of the plating may be expected, therefore, to vary at a rate less than the rate of increase in the bulkhead height. Under these conditions the assistance rendered the stiffeners by the plating, acting through the upper and lower boundaries, will be relatively greater for a low bulkhead than for a high one.

CHAPTER VI

COMMENT AND CONCLUSIONS

General Bulkhead Design.—During the past 30 years a number of interesting and valuable papers on the subject of bulkhead design have been published, both in this country and abroad. A résumé of the most outstanding of these has been given in Appendix D. A striking feature of all these papers is the absence of strain-gage observations, the only references to such data being made in Mr. King's paper¹ and in Prof. Welch's discussion thereon. Such mathematical analyses of the behavior of bulkhead stiffeners as were attempted by the writers of these papers were based on deflection data only. It making an analysis on such a basis one is forced to make unsupportable assumptions as to degree of end fixation of stiffeners and amount of plating acting with the stiffeners.

The strain-gage data taken in connection with the tests reported on herein, although not as consistent as might be desired, give a definite basis for the calculation of degrees of end fixation of stiffeners and the widths of plating acting with stiffeners. The faired strain-gage observations, considered in conjunction with the deflection measurements, and with the resistances of the bulkheads to failure, lead to the following conclusions as to general design:

(1) Up to the elastic limit of the material of the stiffeners, strips of plating, about 60 times the plate thickness in width, act with the stiffeners.

(2) Beyond the elastic limit of the material of the stiffeners, the bulkhead plating comes more and more into play, greatly increasing the ultimate resistance to failure of the bulkhead. The effect of the plating is greater the less the length of the stiffeners.

(3) Riveted or welded stiffeners with well-designed end connections, attaching to substantial structure, act as beams almost

¹ KING, J. FOSTER, "Strength of Watertight Bulkheads," *Trans. Inst. Naval Arch.*, 1916.

completely fixed at the ends. The degree of end fixation is affected materially by the efficiency of the end connections.

(4) The brackets should be of such height that the stress in the stiffener at the bracket apexes will not exceed that at mid-length of the stiffener. The tests have demonstrated the soundness of the well-known rule that the height and depth of brackets should be three times the depth of stiffeners, but not less than one-tenth their length.

(5) Bulkhead plating as thin as 0.15 in. (6 lb.) is strong enough to resist heads in excess of 50 ft. with a stiffener spacing of 4 ft. and with a stiffener length of not over 12 ft.

(6) The brackets of welded stiffeners act as integral parts of the stiffeners, thus increasing the stiffness of the latter. This action is less marked or entirely absent in the case of riveted stiffeners, fitted with the usual types of brackets.

(7) Permanent sets are much greater for riveted stiffeners than for welded ones of generally similar design.

With riveted construction it is difficult and expensive to so dispose the material of the stiffeners as to make full use of the plating acting with the stiffeners. That the neutral axes of stiffeners are ordinarily well over toward the plating is shown by the fact that on merchant ships single-riveted bulkhead seams of less than 50 per cent efficiency have stood up under conditions of maximum load.

Welded construction, on the other hand, lends itself readily to economical disposition of the stiffener material. A scrutiny of columns (10) and (12) of Table 5 will show how much better balanced were the welded stiffeners of the test bulkheads than the riveted stiffeners.

On riveted bulkheads the thickness of plating is determined on considerations of tightness rather than strength. Welded bulkheads, being inherently superior in tightness, may logically be fitted with plating based principally on requirements of strength and of corrosion resistance, the resulting loss in rigidity of the plating being more than offset by the far greater rigidity of the welded stiffeners.

It is reasonable to discount largely the great resistances of the test bulkheads to failure. Doubtless much earlier failures would have occurred had the bulkheads been 20 ft. or more square. Nevertheless, even for bulkheads of great height and width, the assistance rendered by the plating after the elastic

limit of the stiffener material has been passed must be considerable. It is proper, therefore, to use rather low factors of safety in designing bulkheads that will be required to withstand pressures only under emergency conditions.

For riveted water- or oil-tank bulkheads, however, the safety factors must be large, otherwise leakage will occur owing to slip in the joints. On the other hand, moderately small safety factors can be used for welded tank bulkheads without sacrifice of tightness.

The foregoing comment is summed up in the following design recommendations, with the full knowledge of the author that their acceptance will lead to the use of scantlings considerably lighter than those permitted by the classification societies and by present naval practice:

(1) Design bracketed stiffeners on the assumption of complete end fixation, provided the surrounding structure is sufficiently rugged to give adequate support to the brackets. In case this structure is lacking in strength or rigidity, estimate the end fixation factor (it will probably not be less than 0.9) for use in the design of stiffeners and brackets.

(2) In the design of stiffeners assume that strips of plating 60 times the plate thickness in width act with the former.

(3) With stiffener spacing of not over 4 ft. and for heads up to 30 ft., use on welded bulkheads plating of 6 to 10 lb., depending on the degree of corrosion resistance and of stiffness desired. For heads over 30 ft., a minimum plate thickness of $7\frac{1}{2}$ lb. is recommended.

(4) Use the following factors of safety:

	Riveted construction	Welded construction
Ordinary watertight bulkheads	2	2
Oil- or water-tank bulkheads..	5	3

Design Features of Welded Bulkheads.—The tests described herein have brought out certain interesting points as to welding design in addition to the more general conclusions cited.

In the first place, the deposited metal appears to act with the balance of the structure until failure of the latter occurs. The tests have shown clearly, however, that welds must be of suffi-

cient strength to take the full load which may come on them. For example, the welded end connections of bulkhead E broke when overloaded, but did not yield as similar riveted end connections probably would have done. Also the relatively weak type 1 brackets ruptured on all bulkheads so fitted. Experience has shown that weak riveted brackets almost always yield rather than break. The rigidity of welded joints necessitates great care in their design, particularly in locations where concentrations of stress may be encountered owing to sudden changes in the direction of lines of stress.

The fact demonstrated by this series of tests that strips of plating about sixty times the plate thickness in width act with stiffening members may be used to great advantage in the design of most welded plated structures. For example, by eliminating the upper flanges of deck beams and increasing the lower flanges, greater section moment of inertia and modulus are obtained, with the result that the deck is made stiffer, stronger, and lighter.

An interesting welding design is exemplified in the butt joint of type 4 bracket, Fig. 16. This butt was located approximately at the neutral axis of the stiffener and ended near the point of zero bending moment. The position of the joint was therefore such that it was not highly stressed even under the maximum test head. With careful design it is frequently possible to so locate welded connections that they will not lie in zones of heavy stress.

In spite of the admittedly low resistance of welded joints to bending and shock, this deficiency is not of great moment in so far as the application of welding to ships' bulkheads and similar plated structures is concerned. The deposited metal used in making the joints of such a structure cannot be bent through an appreciable angle until the structure itself is so distorted as to be near collapse. Moreover, any shocks coming upon the welds of the structure are transmitted ordinarily through the elastic material of which it is made. The special shock tests included herein prove the ability of welded structures to sustain without damage shocks of an extremely violent nature.

By no means all of the design features of the welded test bulkheads proved satisfactory. The major fault lay in the design of types 1 and 2 brackets. When the sharp knuckle at bracket apex was eliminated in types 3, 4, and 5, much more

satisfactory results were obtained. Another questionable design feature was the placing of plate laps adjacent and parallel to stiffeners. Although the material of such laps served to raise the stiffener section moduli, the welds were otherwise disadvantageously located. Two of the laps in this position opened up under heavy pressure heads, but only after the adjacent brackets had failed.

The welded lug end connections of bulkhead E proved too weak, but there is no reason why such connections should not be successful. Sufficient welding to develop the full strength of the stiffener flanges could be provided by widening these flanges at the stiffener ends; in other words, by fitting brackets parallel to the bulkhead.

All of the other welding designs incorporated in the various bulkheads were satisfactory under maximum test loads. In Appendix E, detailed analysis of these designs is made. It will be observed that ultimate strength figures recommended earlier in this paper are used in the analysis. The behavior of the various welded joints throughout the series of tests demonstrated that these strength figures are not too high and may be used with confidence in the design of bulkheads and similar plated structures.

Both types of boundaries used on welded bulkheads were found practical. The T-bar type is recommended for locations where welding to surrounding structure is not permissible. In other locations the plate type of boundary is preferable.

The welded plate collars around channels piercing one of the bulkheads were tight under heads in excess of 60 ft. except for a leak around one of the channels. It should be noticed that the location of the collars was most unfavorable. Similarly, located riveted staples would probably have leaked to a much greater extent. There seems to be no doubt that welded collars are far superior to the more conventional riveted staples.

The welds around the pipes piercing one of the bulkheads were absolutely tight and were evidently as strong as necessary. The elimination of bolted bulkhead fittings made possible by the substitution of welds is a new departure in ship construction, so far as the author knows, and it is a most desirable one.

Reliability of Welding.—One often hears the argument advanced by those opposed to counting on welding for strength that it is not possible to insure proper workmanship. The

results of the tests reported on are a large-scale object lesson as to just how reliable well-designed welds are when they are made by qualified welders, closely supervised, and provided with proper material and equipment. We have here something over 500 hr. of welding performed during night and day shifts, much of it inside of a closed tank. Still when the welded structures were tested to destruction there was not a single failure that could be attributed to faulty workmanship.

Final Recommendation.—It is the opinion of the author that electric welding performed by competent men under close design and procedure control is fully as reliable as riveting. So confident is he of this that he does not hesitate to recommend the use of welding as a complete substitute for riveting on transverse watertight and oiltight bulkheads of yachts, merchant ships, and naval ships. The adoption of this recommendation will lead to very substantial savings in weight and cost and to improved tightness and stiffness without sacrifice of strength or reliability.

APPENDIX A

WELDING DATA

	Volts at machine	Volts at arc	Amperes
Bulkhead A, 10.2 lb.			
Butt seam.....	65	16	140
Lap seam.....	65	17	160
Stiffeners.....	60	18	150
Boundary bar.....	60	19	160
Welding wire used, 85 lb.; deposited, 68 lb.			
Bulkhead B, 7.65 lb.			
Lap-weld seams.....	60	17	128
Stiffeners.....	60	17	140-150
Boundary plate.....	60	17	150
Welding wire used, 110 lb.; deposited, 88 lb.			
Bulkhead C, 6 lb., flat			
Lap-weld seams.....	60	17	128
Stiffeners.....	60	17	140
Boundary plates.....	60	17	150
Welding wire used, 90 lb.; deposited, 72 lb.			
Bulkhead D, 6 lb., paneled			
Lap-weld seams.....	60	17	128
Stiffeners.....	60	17	140
Boundary plate.....	60	17	150
Welding wire used, 100 lb.; deposited, 80 lb.			
Bulkhead E, 10.2 lb., no brackets			
Lap-weld seams.....	60	17	150
Stiffeners.....	60	17	150
Boundary bar.....	60	17	160
Welding wire used, 75 lb.; deposited, 60 lb.			
Bulkhead F, 7.65 lb., reversed			
Lap-weld seams.....	60	17	140
Stiffeners.....	60	17	150
Boundary plates.....	60	17	150
Welding wire used, 100 lb.; deposited, 80 lb.			
Bulkhead G, 7.65 lb., with interferences			
Lap-weld seams.....	60	17	140
Stiffeners.....	60	17	150
Boundary plates.....	60	17	150
Welding wire used, 115 lb.; deposited, 92 lb.			

All electrodes were bare wire. Electrode sizes were: First layer of butt weld, bulkhead A, $\frac{1}{8}$ in.; stiffeners of bulkheads F and G, $\frac{3}{16}$ in.; all the rest of the welding, $\frac{5}{32}$ in.

PRODUCTION DATA ON WELDING

Bulkhead	Weight of plating, pounds	Approximate feet of welding	Hours required	Weight of wire deposited	Pounds of welding wire per hour	Feet per hour	Feet per pound
A.....	10.20	250	64	68	1.060	4.06	3.82
B.....	7.65	370	85	88	1.035	4.35	4.21
C.....	6.00	370	78	72	0.920	4.74	5.13
D.....	6.00	370	80	80	1.000	4.62	4.62
E.....	10.20	290	54	60	1.110	5.37	4.83
F.....	7.65	370	88	80	0.910	4.21	4.62
G.....	7.65	410	98	92	0.940	4.19	4.45
Totals.....		2,440	547	540	0.986	4.47	4.52

QUALIFICATION TESTS OF WELDERS

(Pounds per square inch)

Name	Position on list of 23 welders	Flat	Vertical	Overhead	Average
Williams.....	3	61,810	57,070	51,400	56,760
Pool.....	9	60,220	57,337	58,220	58,556
Shortridge.....	10	57,930	51,520	44,601 ¹	51,350
Willy.....	17	62,760	45,540 ²	60,405	56,235
Knight (a).....	1	62,071	57,877	51,560	57,167
Knight (b).....	1	60,510	56,824 ¹	44,668	54,007
Webb, 3rd cl. app..	..	41,207 ³	38,847 ³	36,667 ⁴	38,874
Required, min.....	..	45,000	45,000	45,000	52,000

¹ Poor fusion.² Bottom of weld not filled in.³ Dirty weld.⁴ Burnt metal.

Qualification Requirements.—Each welder shall make a single-V butt weld 18 in. long in a $\frac{3}{8}$ in. medium-steel plate; 6 in. shall be welded in the flat position, 6 in. in the vertical position, and 6 in. in the overhead position. The welder shall use the same equipment in the test that he uses on the work. The speed of welding during test shall be that expected on production work. Welds shall be made in one or two layers, slightly reinforced, and the welder may use the size of welding rod and current values which he considers most suitable. Welder may arrange his own work. Backing strip may be used at the bottom of the V. The weld metal at the bottom of the V must indicate thorough penetration and must be relatively uniform in contour. Three

strips, each $1\frac{3}{4}$ in. wide, shall be cut out with gas torch. The weld and edges of strips shall be ground flush, the width being reduced to $1\frac{1}{2}$ in. for 1 in. on each side of weld. The specimens shall then be pulled in a testing machine. The minimum tensile strength required for any specimen shall be 45,000 lb. per square inch, and the average for the three specimens shall be 52,000 lb. per square inch.

APPENDIX B

PHYSICAL TESTS OF MATERIAL USED IN CONSTRUCTION OF TEST BULKHEADS

Miscellaneous data	6-lb. plate	7½-lb. plate	10-lb. plate	Web of 6 × 3½ × 3½ in., 15 lb. channel	15-lb. plate
Size in inches.....	1.5 × 0.186	1.5 × 0.204	1.5 × 0.268	1.5 × 0.350	1.5 × 0.404
Length in inches.....	8	8	8	8	8
Yield point in pounds.....		13,900	16,890	23,360	20,460
Breaking load in pounds.....	17,970	19,270	27,010	34,520	38,230
Elongation in inches.....	1.91	2.47	2.18	1.75	2.19
Dimensions of fracture, inches.	1.142 × 0.131	1.145 × 0.128	1.135 × 0.197	1.151 × 0.253	1.117 × 0.284
Area of fracture in square inch.	0.149	0.142	0.223	0.290	0.317
Elongation, per cent.....	23.7	30.7	27.2	21.7	24
Reduction of area, per cent...	46.5	53.5	44.5	44.7	47.6
Yield point, pounds per square inch.....		42,156	42,614	44,495	33,362
Ultimate tensile strength, pounds per square inch.....	64,408	62,973	67,189	65,752	63,087

CHEMICAL ANALYSES OF MATERIAL USED IN CONSTRUCTION OF TEST
BULKHEADS GIVEN IN PERCENTAGES

Element	6-lb. plate	7½-lb. plate	10-lb. plate	Web of 6 × 3½ × 3½ 15-lb. channel	15-lb. plate	Tee- bar
Carbon.....	0.250	0.236	0.380	0.324	0.347	0.290
Phosphorus....	0.010	0.010	0.010	0.010	0.010	0.010
Sulphur.....	0.038	0.032	0.029	0.037	0.032	0.041

APPENDIX C

The following is a comparison of bending moments and deflections of a beam uniformly loaded and one with uniformly increasing load:

Consider a beam, fixed at the ends, of length 1, and carrying a load increasing from zero at the left end to w_1 per unit of length

at the right end. Let the left support be F_0 , right support F_1 , bending moment at left end M_0 , that at right end M_1 . Take the origin at the left end.

Load at any point $x = wx$

$$\text{Total load } \frac{wl^2}{2} = F_0 + F_1$$

$$M = M_0 + F_0x - \frac{wx^3}{6}$$

$$EI \frac{d^2\delta}{dx^2} = M_0 + F_0x - \frac{wx^3}{6}$$

$$EI \frac{d\delta}{dx} = M_0x + \frac{F_0x^2}{2} - \frac{wx^4}{24} + C_1$$

$$\frac{d\delta}{dx} = 0 \text{ when } x = 0 \quad \therefore C_1 = 0$$

$$\frac{d\delta}{dx} = 0 \text{ when } x = l \quad \therefore M_0 = -\frac{F_0l}{2} + \frac{wl^3}{24}$$

$$EI\delta = \frac{M_0x^2}{2} + \frac{F_0x^3}{6} - \frac{wx^5}{120} + C_2$$

$$\delta = 0 \text{ when } x = 0 \quad \therefore C_2 = 0$$

$$\delta = 0 \text{ when } x = l \quad \therefore M_0 = -\frac{F_0l}{3} + \frac{wl^3}{60}$$

$$F_0 \frac{3}{20}wl^2 \qquad F_1 = \frac{7}{20}wl^2$$

$$M_0 = -\frac{wl^3}{30}$$

$$M = -\frac{wl^3}{30} + \frac{3}{20}wl^2x - \frac{wx^3}{6}$$

$$EI\delta = -\frac{wl^3x^2}{60} + \frac{3}{120}wl^2x^3 - \frac{wx^5}{120}$$

$$M_1 = -\frac{wl^3}{20}$$

$$\text{When } x = \frac{l}{2}, M = +\frac{wl^3}{48} \text{ and } EI\delta = -\frac{wl^5}{768}$$

The following comparison is made between the foregoing and a fixed-ended beam carrying a uniform load of $\frac{wl^2}{2}$:

	Uniformly increasing load	Uniform load
Bending moment at ends.....	$-\frac{wl^3}{30}$	$-\frac{wl^3}{24}$
	$-\frac{wl^3}{20}$	
Average bending moment at ends....	$-\frac{wl^3}{24}$	$-\frac{wl^3}{24}$
Bending moment at center.....	$+\frac{wl^3}{48}$	$+\frac{wl^3}{48}$
Maximum positive bending moment..	$+1.029\frac{wl^3}{48}$	$+\frac{wl^3}{48}$
Deflection at center.....	$-\frac{wl^5}{768}$	$-\frac{wl^5}{768}$

The conclusion is that for hydrostatic loads above the top of a bulkhead the stiffeners may be considered as uniformly loaded beams, provided the calculated bending moments at and near the ends be treated as average rather than absolute values.

APPENDIX D

Résumé of Outstanding Papers on Bulkhead Design.—In 1898 Naval Constructor Woodward published in the *Transactions* of the Society of Naval Architects a paper entitled, "Test of Strength of a Longitudinal Bulkhead Separating Two Engine Rooms." This paper gave the deflections observed during the bulkhead test and described means adopted for reinforcing the stiffener-end connections in order to bring the bulkhead up to the specified requirements. No mathematical analysis of the data was attempted, but the author called attention to the lower deflections for stiffeners adjacent to boundary bulkheads than for other stiffeners.

The following year in the proceedings of the same society H. F. Norton attempted a mathematical analysis of the deflection data that had been reported by Mr. Woodward. In Mr. Norton's analysis the stiffeners were assumed to act as fixed-ended beams supporting loads of the width of the spacing between stiffeners. The author proposed the injection of an arbitrary constant to bring the observed data into agreement with the calculations.

In a second paper published in the *Transactions* of the Society of Naval Architects and Marine Engineers in 1905, Mr. Norton pointed out that the deflection of bulkhead plating must be held to a very small amount to prevent danger of breaking the caulking. The author proposed that bulkhead stiffeners be designed as fixed beams and advocated a maximum allowable fiber stress of 30,000 lb. per square inch, a strip of plating about forty times its thickness (three times the width of stiffener flange) being assumed to act with each stiffener. In the discussion following presentation of Mr. Norton's paper, Professor William Hovgaard objected to the proposal that an arbitrary constant be applied to observed deflections to make them agree with calculated values. He also criticized as illogical and wasteful of material a design embodying stiffeners on both sides of the bulkhead.

In 1910 Prof. William Hovgaard published in the *Transactions* of the Society of Naval Architects and Marine Engineers an extremely valuable paper entitled, "An Analysis of Tests of Watertight Bulkheads with Practical Rules and Tables for their Construction." Professor Hovgaard gave the results of numerous tests in which deflection measurements had been taken, and compared the so-called elastic deflections (differences between total deflections and permanent sets) with curves calculated on the assumption that the stiffeners were fixed-ended beams. The comparisons showed fair agreement and were offered as the justification of certain design rules and tables that were included in the paper. The author recommended the use of uniformly spaced stiffeners not over 4 ft. apart and located on one side of the bulkhead. He also advocated the fitting of lower brackets of a height and width three times the depth of the stiffeners, but not less than one-tenth of their length, and the fitting of upper brackets about 10 per cent shorter and narrower than the lower ones. Professor Hovgaard suggested that the plating be not less than 7-lb. material at the waterline and that 1 lb. be added for each 4 ft. depth below the water line. The author's calculations of section moduli of stiffeners were based on the assumption that there acted with each stiffener a strip of plating of about the area of the flange of the stiffener.

In a paper entitled, "Strength of Watertight Bulkheads," Institution of Naval Architects, 1916, J. Foster King described the 1915 tests conducted by the Bulkhead Committee of the

British Board of Trade. Mr. King gave the deflections of a number of stiffeners and showed that the deflections tapered off from a maximum at the center of the bulkhead to a minimum at the sides. The author expressed the opinion that the function of stiffeners is not so much to provide ultimate strength as to restrict early deflection. He stated, however, that on the bulkheads tested by the committee total deflections as high as $\frac{1}{165}$ of the length of stiffeners did not affect the riveting and calking. Mr. King stated that strain-gage readings taken during the tests were found to be valueless because of derangement of the gages. In the discussion of the paper, Professor J. J. Welsh pointed out that strain-gage readings taken on stiffener webs during the Bulkhead Committee tests had shown that as the head increased the neutral axis moved over toward the bulkhead plating until it finally reached a point which indicated that the entire sheet of plating was acting with the stiffeners.

In 1921 A. M. Robb published his "Notes on Deflection of Bulkheads and of Ships," *Transactions* of the Institution of Naval Architects. In this paper the author attempted to reconcile with the beam theory the deflection data taken by the Bulkhead Committee. He assumed that the stiffeners acted as beams 60 per cent restrained at the ends and that strips of plating of a width of twenty-five times the plate thickness acted with the stiffeners. In the discussion of Mr. Robb's paper much difference of opinion was expressed as to the justification for the assumptions as to degree of end fixation and amount of plating acting with stiffeners. It was pointed out that any number of solutions could be made by varying the assumptions on these two points.

APPENDIX E

Analysis of Design of Welds. *General Note.*—Only typical examples will be considered. The ultimate tensile strength of butt welds will be taken as 36,000 lb. per sq. in. of cross-section. For lap welds loaded normal to lines of welding, the ultimate strength in combined tension or compression and shear will be taken as 36,000 lb. per square inch of throat section. For lap welds loaded parallel to lines of welding, the ultimate strength in shear will be taken as 29,000 lb. per square inch of throat section. The ultimate tensile strength of structural steel will be taken as 60,000 lb. per square inch.

Owing to the thermal effect of the arc on the strength of the material welded, no welded joint will be considered to have an efficiency of over 80 per cent.

All unreinforced fillet welds are assumed to have a face gradient of 45°. By the throat section is meant a plane section through the apex and normal to the face. A reinforced weld has a convex face all points on which are approximately equidistant from the apex.

On the plans of welded bulkheads (Figs. 10 to 16) the leg dimensions in inches of fillet welds are indicated by figures in circles. The letter "R" in a circle with a number signifies that the weld is reinforced. For butt welds thicknesses of welds are given.

Horizontal Plate Seams.—On bulkhead A one of the seams consisted of a lap joint having on each edge a weld of $\frac{1}{4}$ in. leg length, while the other seam consisted of a butt joint $\frac{1}{4}$ in. thick.

We have an efficiency of the lap joint of

$$\frac{2 \times 0.25 \times 0.707 \times 36,000}{0.25 \times 60,000} = 0.85 \text{ (take 0.8)}$$

The butt weld had an efficiency of

$$\frac{0.25 \times 36,000}{0.25 \times 60,000} = 0.6$$

The most severe plate stresses normal to the seams occur in way of the stiffeners. We see from the faired curve of Fig. 50 that stresses at the locations of the seams were only about half of those at mid-height of stiffeners. It appears, therefore, that the efficiencies of both seams were adequate to develop the full strength of the bulkhead plating.

There were no failures in horizontal plate seams of any of the bulkheads tested.

Vertical Plate Laps.—Bulkhead B was provided with vertical plate laps which were located adjacent to stiffeners in order that the material of the laps would contribute to the section moduli of the stiffeners.

Let us consider as a fixed ended beam a horizontal strip of plating 1 in. wide and extending between the stiffeners. We have for this strip

$$\begin{aligned} \frac{I}{y} &= \frac{(0.191)^2}{6} = 0.0061 \\ M_0 &= -\frac{whl^2}{12} = -\frac{61.2 \times 48^2}{144 \times 12}h = -81.5h \\ f &= \frac{81.7h}{0.0061} = 13,400h \text{ lb. per square inch} \end{aligned}$$

Notation is the same as that used in the body of the paper.

It is evident that under the assumed conditions the horizontal tensile stress in the plating adjacent to stiffeners passed the yield point at a very moderate head. After the passing of this point the horizontal stresses in the plating were principally tension.

At various pressure heads applied to bulkhead C the horizontal tension at mid-height of the center span of plating was measured with the strain gages. These measurements indicated that the horizontal tension ranged from about 14,000 lb. per square inch at a 12-ft. head to about 19,000 lb. per square inch at a head of 38 ft. Similar measurements on bulkhead E gave horizontal plate tensions from about 7,000 lb. per square inch at a 12-ft. head to about 10,000 lb. per square inch at a 24-ft. head. At higher heads the measured plate tensions fell off slightly.

The plate tensions in bulkhead B were probably about midway between the values measured on bulkheads C and E respectively.

The edges of the laps of bulkhead B were secured by fillet welds with leg length from $\frac{1}{4}$ to $\frac{5}{16}$ in. The material of these welds was subjected to horizontal stresses corresponding to those in the bulkhead plating and also to heavy vertical stresses corresponding to the bending stresses in the stiffeners. There were also small longitudinal shears.

At the time of the crumpling of the standing flanges of the stiffeners of B bulkhead the vertical plate tension at mid-height of bulkhead was about 25,000 lb. per square inch; and when the stiffeners broke at the apexes of the brackets, the vertical compression in the plating at these points must have been from 30,000 to 35,000 lb. per square inch.

It is probable that at high-pressure heads the weld material in the vertical lap joints was stressed considerably beyond the value taken herein as its ultimate strength.

There were two failures in the welds of vertical laps, but both occurred at points adjacent to brackets that had previously failed.

On both 6-lb. bulkheads C and D the two courses of plating forming the vertical laps separated by over an inch without failure of the welds.

In view of the disadvantageous positions of the welded seams when vertical laps adjacent to stiffeners are used, the author is

of the opinion that it is generally preferable to use horizontal laps, or vertical laps placed well clear of stiffeners.

Lap Joints between Plating and Boundary Bar or Plate.—On bulkhead B $\frac{1}{4}$ -in. reinforced to $\frac{7}{16}$ -in. welds were used to connect the bulkhead plating to the 10-lb. boundary plates.

These joints were so located that they were required to take a certain amount of bending, which was relieved, however, when the edges of the bulkhead plating passed the yield point. Thereafter the joints took moderate tensions corresponding to those in bulkhead plating.

The upper and lower joints in way of stiffeners also took stresses owing to bending moments at the ends of the stiffeners. These stresses were relatively light, however, on account of the reinforcement given by the brackets.

It appears that the thicknesses of the welds were somewhat greater than necessary. Welds with a leg length equal to the thickness of the bulkhead plating should have been adequate.

There were no failures of the joints between plating and boundaries of any of the bulkheads.

Joints between Boundaries and Tank.—On bulkhead G the boundary plates were connected to the tank by $\frac{3}{8}$ -in. reinforced tee welds.

It seems rational to design these welds with sufficient strength to take the maximum bending moment which can be applied through the boundary plates.

$$\frac{\text{Section modulus of throat sections of tee welds}}{\text{Section modulus of boundary plate section}} \geq \frac{60,000}{36,000}$$

For one linear inch of boundary of bulkhead G

$$\begin{array}{ll} \text{Section modulus of throat sections of tee welds} & = 0.10 \text{ about} \\ \text{Section modulus of boundary plate sections} & = 0.01 \text{ about} \end{array}$$

It appears from the foregoing comparison that the welds were redundant in strength in so far as resistance to bending was concerned. They were required to take, in addition, the moderate stresses imposed by the tension in the bulkhead plating and also small shearing stresses. Portions of the boundary tee welds adjacent to the ends of stiffeners require special consideration and will be dealt with in conjunction with the bracket connections. For the remaining portions two continuous fillet welds of length of leg equal to the thickness of the boundary plates would have given ample strength.

There were no failures in any of the tee welds connecting boundary plates to the tank.

Tee Welds Connecting Stiffener Webs and Face Plates.—

These welds formed an integral part of each stiffener, and regardless of the weld cross-sectional areas they must have taken longitudinal tensile stresses corresponding to those existing in adjacent material. They also must have taken longitudinal shear. Even at high heads the magnitude of the latter was small, and it could have been taken care of by light intermittent welds. It seems undesirable, however, to introduce such welds in a location where heavy tensile stresses due to bending occur.

On all of the bulkheads welds of redundant strength were used to connect stiffener webs and face plates. Two continuous fillet welds of $\frac{1}{8}$ -in. leg length would have sufficed.

In the joints under discussion there were no failures except those which were induced by and formed a part of the failure of nearby brackets.

Tee Welds Connecting Stiffener Webs and Bulkhead Plating.

These welds were required to take stresses corresponding to but of a lesser magnitude than those in welds between webs and face plates. The former also had to take bending moments imposed by the bulkhead plating as well as small transverse shearing stresses due to the hydrostatic pressures.

On bulkhead A the stiffener webs were connected to the plating by continuous $\frac{3}{8}$ -in. fillet welds. The section modulus of a linear inch of cross-section through the bulkhead plate and through the throat of the weld was about 0.03 against 0.01 for a similar length of the plating. The welds were adequate, therefore, to take any bending imposed by the plating.

Brackets.—The type 4 brackets of bulkhead G were connected to the tank by $\frac{7}{16}$ -in. tee welds $18\frac{5}{8}$ in. long. It seems reasonable to assume that a corresponding length of the boundary plate tee welds acted with each bracket. For a type 4 bracket, the modulus of the throat sections of $18\frac{5}{8}$ in. lengths of tee welds between bracket and tank and between boundary plate and tank was about 81. The section modulus of the stiffener with a strip of plating of width 60 times its thickness was only 10.1. On the assumption that the stiffeners acted as fixed-ended beams the end bending moment was twice that at mid-height. The section modulus of the tee welds at the end of the stiffener

should have been not less than $10.1 \times 2 \times \frac{60,000}{36,000} = 33.8$ It

appears, therefore, that the welds were heavier than was necessary to develop the full strength of the stiffeners. The other types of brackets used were not as deep as the type 4, and the welds connecting these brackets to the tank were more heavily stressed than were those of type 4 brackets.

Among all of the bulkheads tested there was only one failure of the tee welds between tank and brackets. This occurred on one of the type 2 brackets of A bulkhead as a result of the suddenly applied load which occurred when both of the type 1 brackets ruptured. In this case the welds showed evidence of poor penetration.

The complete failure of all of the type 1 brackets may be attributed to the points of discontinuity in strength existing at the apexes of these brackets. Type 2 brackets were a decided improvement in this respect. A further improvement was effected in brackets of types 3, 4, and 5 by shaping the stiffener face plates to an easy curve at bracket apexes.

General Remarks.—In the foregoing discussion a number of the welds as made have been pronounced redundant in strength. In general the welds were actually made slightly heavier than called for by the design as the inspectors would not pass welds lighter than those shown on the plans. In all of the welds that were required to take bending (the tee welds of boundary plates, for example) small reductions in weld dimensions would have caused great decreases in strength.

PART IV

THEORY AND APPLICATION OF THE BASE PLATE
ARC-WELDED RAIL JOINT

BY

FRANK B. WALKER

CHAPTER I

GENERAL

A lessened annual depreciation of \$15,000,000 would result if 5 years could be added to the average life of the 25,000 miles of street railway track in paved streets, according to estimates by way engineers. A complete solution of the welded rail-joint problem would accomplish this result.

Very little attention in general has been paid to the welding of unpaved track on street railways and none to the welding of track on steam railroads. The two principal reasons why most of the street-railway or steam-railroad engineers have not attempted to weld open or unpaved track are their belief that expansion and contraction of the rails because of temperature changes cannot be properly cared for and that welded joints are not strong enough. My experience in welding and maintaining over 200 miles of open track in the rigorous climate of eastern Massachusetts indicates that these objections can be met successfully. Expansion and contraction have been taken care of successfully by installing expansion joints at about 750-ft. intervals.

It will be interesting to consider the size of the field thus opened up, as the 20,000 miles of unpaved street railways have about 6,000,000 rail joints and the more than 250,000 miles of steam railroads have more than 80,000,000 joints. The annual saving in depreciation and maintenance if all these rail joints were welded would be from \$50,000,000 to \$100,000,000.

The present arc-welded rail joint practically follows the lines of the old bolted fish-plate joint with the substitution of welding for the bolts. Several street railways weld a plate to the rail bases in addition to the fish plates, but the purpose of this plate has been only to form a continuous tie plate and to assist the fish plates in some unknown degree, the fish plates being depended upon for strength. In the fish-plate joint the metal lies too near the neutral axis of the section to be economically disposed for stiffness or strength in bending. A structural engineer splices

a steel girder by placing a plate on top and another on the bottom flange. This is the ordinary practice in riveted connections and will undoubtedly be the practice in welded connections in structural work.

I have therefore designed, built, tested, and used a new arc-welded rail joint with top and bottom splice plates arranged closely in accordance with the principles of structural splicing of girders.



FIG. 1.—Open 75-lb. T-rail track welded in 1922.

FIG. 2.—Open 75-lb. T-rail track welded in 1923.

FIG. 3.—Gantlet open 75-lb. T-rail track welded in 1923.

This new joint is constructed with a base plate approximately 7 by $\frac{3}{4}$ by 28 in. welded to the rail bases throughout its length and with two head bars approximately 1 by $1\frac{1}{2}$ by 16 in. welded along their upper edges to the under side of the rail heads, one bar on each side. The lower edges of the head bars are not welded to the rail webs because previous experience indicates that welding anything to the webs of high-carbon rails is very apt to cause them to crack.

CHAPTER II

THEORY OF DESIGN

The fundamental principle that should govern the design of a rail joint is that it should be as nearly as possible of the same strength as the rail itself. To attempt to determine the actual shear and bending stress in the rail and to design the joint in accordance is rather beside the point, because the shear and bending stresses in rails are complicated, the character and amount being indeterminate, as the rigidity of the ties and



FIG. 4.

FIG. 5.

FIG. 4.—A sun kink on welded open 75-lb. T-rail track in June, 1921.
Expansion joints were then installed and no further kinks appeared.

FIG. 5.—Expansion joint developed and used on open welded track.

ballast varies from point to point. With welded joints the temperature stresses become important and are somewhat indeterminate, as temperature fluctuations are not accurately known.

A simple beam supported at the two ends with a vertical load has compressive stress in the top flange and tensile stress in the lower flange. A continuous beam is one supported at more than two points with the loads arranged in any manner. Continuous beams have compression at some places and tension at

others in the same flange. The location and amounts of these stresses in the flange depend upon the number and relative elevation of the supports, the size and arrangement of the loads, and the spacing of the supports. As the loads move, the location



FIG. 6.—Expansion joint developed and used on open welded track.

of the stresses and their amount and character are altered. Welded rails are continuous beams and are subject to different stresses in different locations and in the same location at different times. These stresses vary both in character and amount.

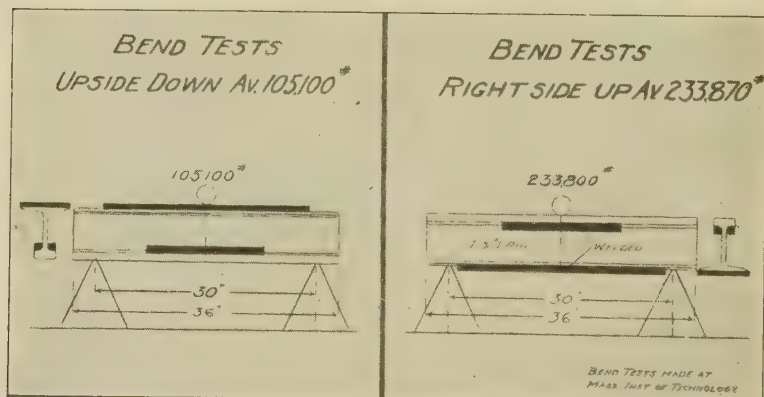


FIG. 7.—Sketch showing method of making bend tests on base-plate joints upside down and right side up.

The welded joint must therefore be capable of taking both tension and compression in both head and base.

Wheel loads of street cars vary considerably. The four-wheel Birney car weighs with its load 28,000 lb., or 7,000 lb. per wheel. The heaviest street railway car with which I have had experience, an eight-wheel car, weighs with its load about 92,000

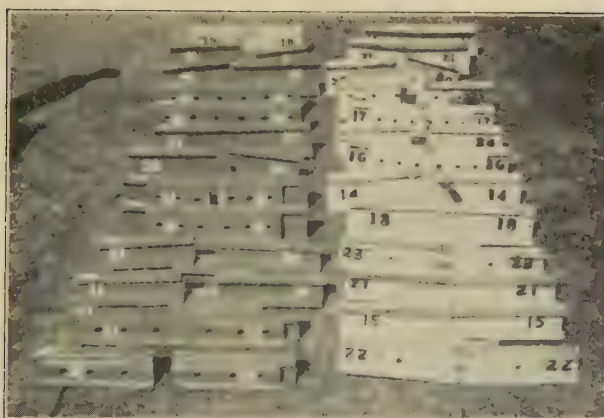


FIG. 8.—Views of 42 of the experimental joints, after testing at Massachusetts Institute of Technology.

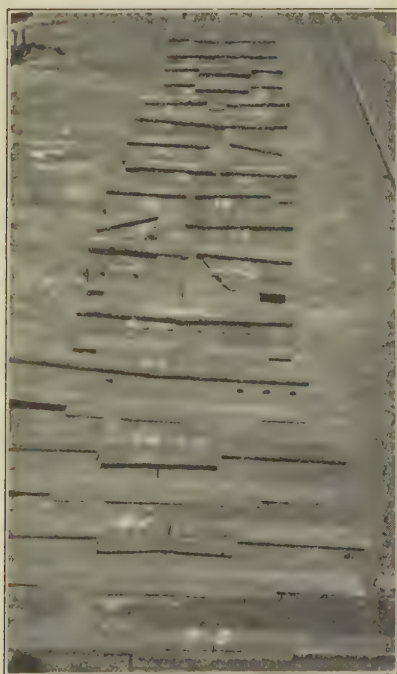


FIG. 9.—Views of 42 of the experimental joints, after testing at Massachusetts Institute of Technology.

lb., or 11,500 lb. per wheel. The heaviest street railway car known has a wheel load of 17,500 lb. Steam-railroad wheel loads run up to 30,000 lb. and even a little more. Computing the tension in a 7-in., 91-lb. T-rail acting as a simple beam under a single wheel load of 11,500 lb. centrally located with supports 30 in. apart, we find that there is about 4,600 lb. per square

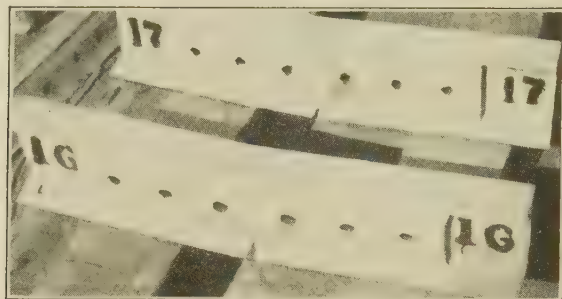


FIG. 10.—Fish-plate joints on 7-in. T-rail.

Joint No. 16, yield point load 145,000 lb. Joint No. 17, yield point load 150,000 lb.

inch tension in the base. This is a very low unit stress. Recent telemeter tests on street-railway rails under actual service conditions confirm this figure. The total tension is about 15,500 lb. in the base, and under unusual service impact would not exceed 31,000 lb.

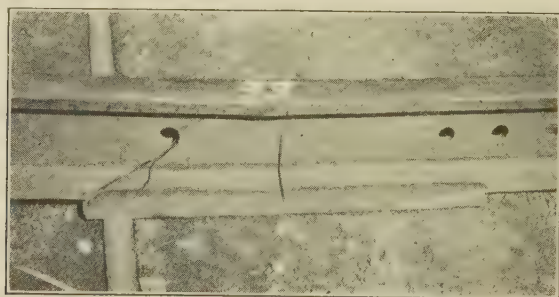


FIG. 11.—Base-plate joint on 7-in. T-rail.

Joint No. 93, broke rail at 264,250 lb. Base plate $\frac{3}{4} \times 7 \times 28$ in.

The compression in the rail or joint owing to increase in temperature is taken care of by actual movement of the rail where expansion joints are provided or by internal compressive stresses where there are no expansion joints or where rails are so embedded in the pavement as to prevent any lateral or longi-

tudinal movement. Expansion joints are not needed, nor do I ever use them in welded rails on paved streets, except on some bridges.

The tension caused by a decrease in temperature is the largest and most important of all stresses to consider in the design of



FIG. 12.—Base-plate joints on 7-in. T-rail.

Joint No. 36, broke rail at 134,350 lb. Joint No. 37, broke rail at 128,100 lb. Base plates $1 \times 7 \times 16$ in.

joints. The amount of this tension depends on several factors, such as the area of rail, temperature range, friction, binding of spikes on rails, etc., but undoubtedly is less than the theoretical amount. As a measure of the influence of the factors, I find the run or movement of expansion joints in open track is rarely more than one-half the theoretical amount. The usual temperature



FIG. 13.—Base-plate joint on 7-in. T-rail.

Joint No. 3, rail broke at 175,000 lb. Base plate $\frac{3}{4} \times 7 \times 16$ in.

range assumed for computation is 100°F. , in which case the tension caused is found as follows:

Coefficient of expansion $\times$ modulus of elasticity or $0.00065 \times 29,000,000 = 18,850$ lb. per square inch.

Total tension for 7-in., 91-lb. rail is $18,850 \times 8.96 = 168,900$ lb.

We are interested in the total computed tension, which is 168,900 lb. caused by temperature, plus tension caused by wheel load and impact, which may be 31,000 lb., or a total of about 200,000 lb. The base plate and head bars have an area of 8.25 sq. in. to resist a tension of 200,000 lb., which gives 24,240 lb. per square inch, or about two-thirds of the elastic limit.

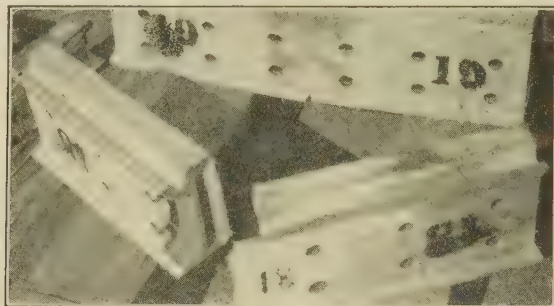


FIG. 14.—Fish-plate joints on 9-in. girder rail.
Joint No. 18, plate broke at 287,500 lb. Joint No. 19, $1\frac{1}{16}$ -in. deflection.

I have installed in unpaved track many thousands of 60-lb. and 75-lb. T-rail fish-plate welded joints. The areas of these joint plates compared to rail area are:

	Area rail, square inches	Area plate, square inches	
		Old bars	New plates
75-lb. T-rail.....	7.4	4.5 to 6.0	
60-lb. T-rail.....	5.9	4.1 to 4.7	

This shows that the area of welding bars in actual service necessary to take temperature and wheel-load tension is only from 60 to 80 per cent of the rail area. My experience is the same on welded 56- and 48-lb. rails.

CHAPTER III

TESTING EXPERIMENTAL JOINTS

I have made, concurrently with my development of theoretical design, 66 experimental joints that have been tested on the 400,000-lb. Riehle testing machine at the Massachusetts Institute of Technology. Test joints were made on three sections of rail; namely, 75-lb. T, A.S.C.E. section $4\frac{13}{16}$ in. high, and 104-lb. girder rail, Bethlehem section 104-401, although the major portion (43) of the tests were on the 7-lb. 91-lb. rail. In each

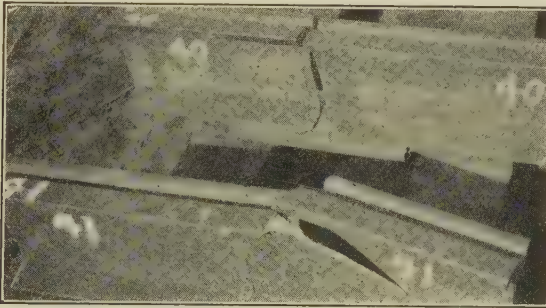


FIG. 15.—Base-plate joints on 9-in. girder rail.

Joint No. 40, weld broke at 263,500 lb. Joint No. 41, rail broke at 197,100 lb.

series there were several joints tested that were made with the standard welded fish plates. These were for comparison with results obtained on the experimental joints. Types of experimental test joints were:

	Number of joints tested
With standard fish plates and no base plates.....	11
With base plates of various sizes without fish plates or head bars.....	43
With base plates and small head bars.....	8
Miscellaneous.....	4
Total.....	66

All joints were welded with the electric-arc metallic electrode process. The electrode was $\frac{5}{32}$ by 14 in. soft-steel welding wire

purchased in the open market. The plates and bars were all of commercial machine steel cut to proper length. Welding was done by five different welders. Care was taken to bring the rail heads into contact, but no pressure was used to cause initial head compression. The uniformly high results of the bend tests

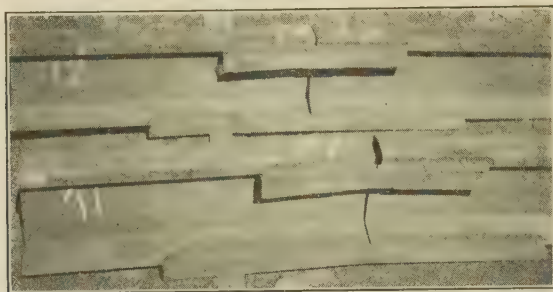


FIG. 16.—Base plate, head-bar joint on 7-in. T-rail tested upside down. No. 91, two $1\frac{1}{4} \times 1\frac{1}{4}$ in. head bars, weld broke at 90,750 lb. No. 92, one $1\frac{1}{4} \times 1\frac{1}{4}$ in. head-bar, weld broke at 52,150 lb.

indicate that initial head compression is unnecessary. Experimental joints in the series 1 to 69 had double-welding bead, and joints in the series 70 to 92 had a single bead. All beads were 1 in. length of bead to 3 in. length of welding wire. Bending tests only were made; in all but three the distance between supports was 30 in., with load applied in the center, and a total

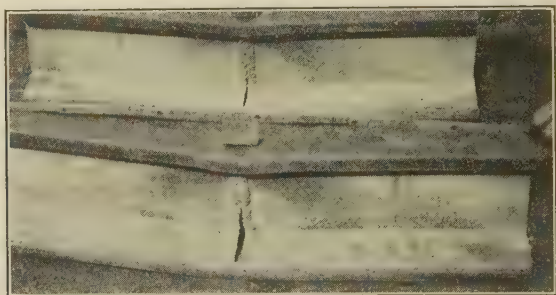


FIG. 17.—Test joints 28 and 29, 7-in. rail 1- x 7- x 36-in. base plate. No. 28, yield point load 303,000 lb. No. 29, yield point load 312,000 lb. Heads crushed $\frac{1}{4}$ in. at maximum load of 358,000 lb. and 362,500 lb., respectively.

length of joints of 36 in. Two short joints were tested with 20 in. between supports and one with the load off center. Many of the joints were painted with a thin cement grout to make a white, brittle coating so that observers could easily detect strains, lines, and cracks in the joints, bars, and welds under

the progressive application of the load. Numerous photographs of joints before and after testing are shown, and a complete tabulation of tests is given in Table 3.

TABLE 3.—BEND TESTS ON EXPERIMENTAL WELDED RAIL JOINTS
7-in., 91-lb., Section 282 T-rail, 30-in. Span except Joints 1 and 5

Joint number	Type of joint	Size of plate, inches	Yield load, pounds	Maximum load, pounds	Kind of failure
16	Fish plates	2— $\frac{7}{8}$ × 26	145,000	159,000	No break
17	Fish plates	2— $\frac{7}{8}$ × 26	150,000	177,000	No break
34	Fish plates	2— $\frac{7}{8}$ × 26	184,800	184,800	1 plate broke
35	Fish plates	2— $\frac{7}{8}$ × 26	155,200	155,200	Both plates broke
Average.....	Fish plate		158,750	169,000	
3	Base plate	$\frac{3}{4}$ × 7 × 16	168,000	175,000	Rail broke
14	Base plate	$\frac{3}{4}$ × 7 × 16	269,200	269,200	Rail broke
36	Base plate	1 × 7 × 16	134,350	134,350	Rail broke
37	Base plate	1 × 7 × 16	128,100	128,100	Rail broke
Average.....	16-in. base plate		174,910	176,660	
50	Base plate	$\frac{1}{2}$ × 10 × 28	206,850	238,700	Weld failed on base plate
51	Base plate	$\frac{1}{2}$ × 10 × 28	239,100	271,100	Base plate broke
52	Base plate	$\frac{1}{2}$ × 10 × 28	218,000	266,100	Base plate broke
53	Base plate	$\frac{1}{2}$ × 10 × 28	250,350	263,350	Weld failed on base plate
59	Base plate	$\frac{1}{2}$ × 10 × 28	245,500	245,500	Weld failed on base plate
61	Base plate	$\frac{1}{2}$ × 10 × 28	268,000	292,650	Weld failed on base plate
Average.....	$\frac{1}{2}$ × 10 × 28" base plate		237,967	262,900	
60	Base plate	$\frac{5}{8}$ × 10 × 28	226,000	226,000	Weld failed on base plate
56	Base plate	$\frac{5}{8}$ × 10 × 28	270,350	270,350	Base of rail broke, also welds failed
57	Base plate	$\frac{5}{8}$ × 10 × 28	284,200	284,200	Weld failed on base
Average.....	$\frac{5}{8}$ × 10 × 28" base plate		260,183	260,183	
53	Base plate	$\frac{3}{4}$ × 7 × 28	241,450	241,450	Weld failed
54	Base plate	$\frac{3}{4}$ × 7 × 28	238,000	242,350	Weld failed
55	Base plate	$\frac{3}{4}$ × 7 × 28	227,000	227,000	Weld failed
Average.....	$\frac{3}{4}$ × 7 × 28" base plate		235,483	236,930	
Average.....	Of all 28-in. plates		242,900	255,700	
28	Base plate	1 × 7 × 36	303,000	362,500	No failure
29	Base plate	1 × 7 × 36	312,000	358,000	No failure
Average.....	1 × 7 × 36" base plate		307,500	360,250	
13	Base plate	$\frac{7}{8}$ × 7 × 16	75,000	75,000	Weld broke
(This joint was tested upside down.)					
24	Butt weld	No plate	132,700	132,700	Weld broke, also through rail
G.E. 2	Base plate	$\frac{7}{8}$ × 8 $\frac{1}{2}$ × 18	235,000	235,000	Weld broke
1	Base plate	$\frac{3}{4}$ × 7 × 10	286,000	286,000	Rail broke; 20" span
5	Butt weld	No plates	177,500	177,500	Weld broke; 20" span

NOTE.—Welds are all double beads.

TABLE 3.—BEND TESTS ON EXPERIMENTAL WELDED RAIL JOINTS.—
(Continued)*7-in. T-rail, 30-in. Span, Single Bead*

Joint number	Type of joint	Size of plate, inches	Yield load	Maximum load, pounds	Kind of failure
70	Base plate	$\frac{3}{4} \times 7 \times 28$	234,600	234,900	Welds failed; poor
72	Base plate	$\frac{3}{4} \times 7 \times 28$	241,750	243,250	Rail broke
77	Base plate	$\frac{3}{4} \times 7 \times 28$	227,250	227,250	Welds failed
79	Base plate	$\frac{3}{4} \times 7 \times 28$	221,900	246,400	Broke rail
80	Base plate	$\frac{3}{4} \times 7 \times 28$	261,900	282,000	Rail broke through holes
93	Base plate	$\frac{3}{4} \times 7 \times 28$	264,250	264,250	Rail broke
76	Base plate	$\frac{3}{4} \times 7 \times 28$	209,900	217,350	Weld failed
78	Base plate	$\frac{3}{4} \times 7 \times 28$	231,200	247,900	Rail broke
71	Base plate	$\frac{3}{4} \times 7 \times 28$	212,150	212,150	Rail broke
Average.....			233,870	241,700	

76 and 71 were first tested upside down, then tested right side up.

7-in. T-rail Joints Tested Upside Down to See What They Would Hold in Case of Reversal of Stresses

Joint number	Type of joint	Yield load	Maximum load	Kind of failure
71	$\frac{3}{4} \times 7 \times 28''$ vertical seam	54,400	54,400	Broke vertical seam
75	$\frac{3}{4} \times 7 \times 28''$ 1— $1\frac{1}{4} \times 1\frac{1}{4}$ in.	60,150	60,150	Bar weld failed
76	$\frac{3}{4} \times 7 \times 28''$ 1— $1\frac{1}{4} \times 1\frac{1}{4}$ in.	59,900	59,900	Bar weld failed
92	$\frac{3}{4} \times 7 \times 28''$ 1— $1\frac{1}{4} \times 1\frac{1}{4}$ in.	52,150	52,150	Bar weld failed
		56,650	56,650	
73	$\frac{3}{4} \times 7 \times 28''$ 2— $1\frac{1}{4} \times 1\frac{1}{4}$ in.	122,300	122,300	Bar weld failed
74	$\frac{3}{4} \times 7 \times 28''$ 2— $1\frac{1}{4} \times 1\frac{1}{4}$ in.	114,700	114,700	Bar weld failed
90	$\frac{3}{4} \times 7 \times 28''$ 1— $1\frac{1}{4} \times 1\frac{1}{4}$ in. and 1— 13'' section of welding bar	92,650	107,050	Broke rail
91	$\frac{3}{4} \times 7 \times 28''$ 2— $1\frac{1}{4} \times 1\frac{1}{4}$ in. and 1— 13'' section of welding bar	90,750	90,750	Weld failed in bars
		105,100	108,700	
One compromise 7 to 9 in. with three base plates				
81	This tested with 32-in. span	269,750		Yield and failure of weld; broke 7-in. rail also

TABLE 3.—BEND TESTS ON EXPERIMENTAL WELDED RAIL JOINTS.—
(Continued)

9-in. Section 104-401 Girder Rail, 30-in. Span

Joint number	Type of joint	Size of plate, inches	Yield load	Maximum load	Kind of failure
18	Fish plate	2— $\frac{7}{8}$ × 36	285,000	287,500	Plates broke
19	Fish plate	2— $\frac{7}{8}$ × 36	225,000	313,000	No break, $1\frac{1}{4}$ " deflection
38	Fish plate	2— $\frac{7}{8}$ × 36	222,700	222,700	Plates broke
39	Fish plate	2— $\frac{7}{8}$ × 36	273,300	273,300	Load applied 4" off center; plates broke
Average.....	Fish plate, omit No. 39		244,200	274,400	
Average.....	Fish plate, with No. 39		251,500	274,100	
11	Base plate	1 × 7 × 16	240,000	240,000	Weld failed
12	Base plate	1 × 7 × 16	292,000	292,700	Weld failed
20	Base plate	1 × 7 × 16	256,800	256,800	Rail broke
41	Base plate	1 × 7 × 16	197,100	197,100	Rail broke
Average.....	Base plate		246,500	246,500	
40	Base plate, no head contact	1 × 7 × 16	$\frac{1}{4}$ " gap closed at 162,000	263,500	Weld failed
G.E. 3 9" girder	Base plate, no head contact	$\frac{7}{8}$ × 8 × 18	255,000	309,000	Weld broke

75-lb. A.S.C.E. T-rail, 30-in. Span

Joint number	Type of joint	Size of plate, inches	Yield load	Maximum load	Kind of failure
22	Fish plate	2— $\frac{3}{4}$ × 24	77,600	77,600	Did not break
23	Fish plate	2— $\frac{3}{4}$ × 24	81,400	81,400	Did not break
30	Fish plate	2— $\frac{3}{4}$ × 32	82,700	82,700	Broke plates
Average.....	Fish plate		80,600	80,600	
2	Base plate	$\frac{3}{4}$ × 7 × 16	113,000	130,500	Weld failed
15	Base plate	$\frac{7}{8}$ × 7 × 16	124,000	124,700	Weld failed
21	Base plate	$\frac{7}{8}$ × 7 × 16	120,000	120,400	Weld failed
32	Base plate	$\frac{3}{4}$ × 7 × 16	99,000	99,500	Weld failed
33	Base plate	$\frac{7}{8}$ × 7 × 16	96,000	97,900	Weld failed
Average.....	Base plate	7 × 16	110,400	114,600	
25	Base plate	$\frac{7}{8}$ × 5 $\frac{1}{2}$ × 26	140,000	152,400	No failure
26	Base plate	$\frac{7}{8}$ × 5 $\frac{1}{2}$ × 26	130,000	143,700	No failure
Average.....	Base plate	5 $\frac{1}{2}$ × 26	135,000	148,000	
6 75-lb. T	Base plate and butt weld entire section	$\frac{3}{4}$ × 6 × 6	81,000	81,000	Rail broke
G.E. 4 75-lb. T	Base plate and butt weld entire section	1 × 8 × 18	135,000	152,800	Rail broke
31 75-lb.	Fish plate	2— $\frac{3}{4}$ × 32	42,000	42,000	Weld broke
Fish plates were	cut at center and rewelded				

No accelerated or repeated impact tests were made, as I do not believe that such tests are of value. So far as I have observed no properly welded rail joint (including many thousands of

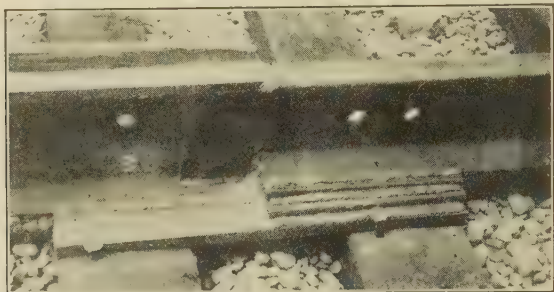


FIG. 18.—Compromise joint 7- to 9-in. rail.
Cost \$6 complete in place.

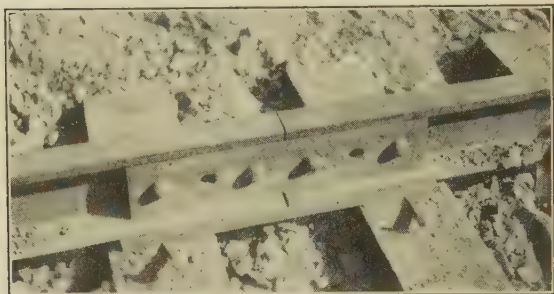


FIG. 19.—Showing method of assembling base-plate joint in track.
A steel insert is placed between rail heads before welding to make head contact.

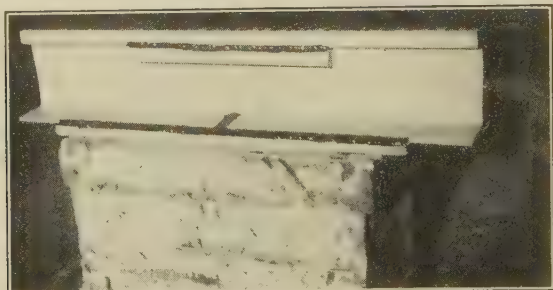


FIG. 20.—Sample base-plate joint on 7-in. T-rail.
Exhibited at American Electric Railway Association in Cleveland, October, 1927.

thermit, Lorain bar, and arc-welded types) put in track under my direction has failed on account of wheel impact. The heads of all welded rail joints in track are ground smooth with a rotary or reciprocating brick grinder.

In making the bend tests several of the joints were tested upside down, both to determine the strength of such joints under the wave-like motion that occurs in track and that causes reversal of stresses and to determine the amount of the tension and horizontal shear in the welded seams. Two of the joints were tested

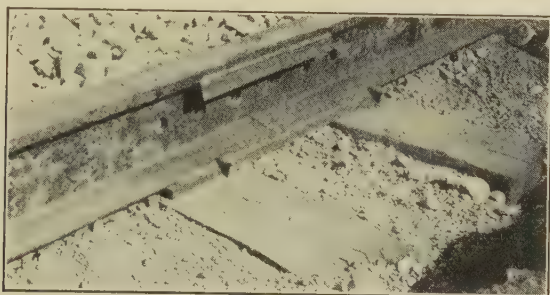


FIG. 21.—Base-plate, head-bar joint installed on 7-in. rail.

upside down until the head-bar seams failed, after which the joints were turned in the testing machine and tested right side up.

I made no direct tension tests on the joints for the following reasons: The strength of joints in tension depends on the tensile strength of the splice plates and shearing strength of the welded seams. The strength of machine steel is well established, and

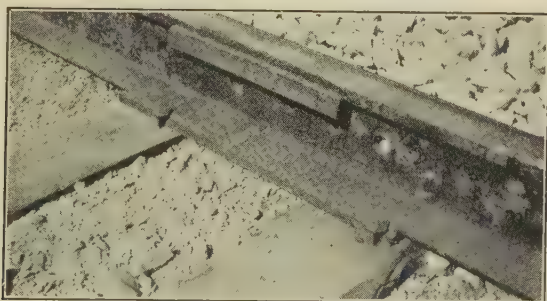


FIG. 22.—Base-plate, head-bar joint installed on 7-in. rail.

the shearing strength of welded seams has been quite thoroughly tested by other investigators. However, the bend tests of the experimental joints, both right side up and upside down, gave an opportunity to approximate closely the horizontal shear in the welded seams of the test joints under vertical loads. I have computed from these tests by dividing the bending moment by the distance between centers of plate and bars that the base

seams carried 311,700 lb., or 11,100 lb. per lineal inch, and the head seams 140,000 lb., or 8,800 lb. per lineal inch. With both head bars and base plate together in tension they would resist a total direct tension of 451,700 lb. A summary of this is given in Table 1.

TABLE 1.—AVERAGE OF THE BEND TESTS ON THE 7-IN. RAIL JOINTS, 30-IN. SPAN

	Number	Total load	
		Pounds, yield point	Pounds, maximum
Fish-plate joints.....	11	158,750	169,000
Base-plate joints:			
Double bead, $\frac{1}{2}$ by 10 by 28 in.....	6	237,967	262,900
Double bead, $\frac{5}{8}$ by 10 by 28 in.....	3	260,183	260,183
Double bead, $\frac{3}{4}$ by 7 by 28 in.....	3	235,483	236,930
Single bead, $\frac{3}{4}$ by 7 by 28 in.....	9	233,870	241,700
For bend tests on base plate joints upside down:			
$\frac{3}{4}$ by 7 by 28-in. base plate, two head bars-	4	105,100	108,700

A few joints made of 75-lb. rail and 9-in. rail only were tested, and it will be noticed from the tabulation of the tests that whereas the yield-point load carried by the fish-plate joint on the 75-lb. rail was 80,600 lb., a base-plate joint without fish plates on 75-lb. rail carried a load of 110,400 to 135,000 lb. The tests on the 9-in. rail with the fish plate joint showed an average yield-point load of 244,200 lb., and the same rail with an 18-in. base-plate joint carried a yield-point load of 255,000 lb. There is little doubt that with a 26-in. base plate results would have been correspondingly greater.

Particular attention is called to the fact that out of the 43 base-plate joints tested, the rails failed in 16 of them. It is evident that the base-plate joint is stronger than the rails.

CHAPTER IV

PRACTICAL APPLICATION

The design having been shown to be adequate by the experimental test joints, I applied the new joint to regular track construction. Almost all new or relaying rail installed recently under my direction in paved and unpaved streets and on open track was welded with the base-plate joints. Since May, 1927, over 1,000 of these joints have been put in, some of them under city service of 30 cars per hour.



FIG. 23.—A 9-in. girder rail laid and welded with base-plate joints.

The base-plate joint in the track is quite simply assembled and welded. When the rails are laid, they are bolted by means of a pair of short assembly plates. The base plate is then slipped under the joint exactly like a tie plate. After the rails are lined and surfaced the base plate is welded, first one end and then the other, so that welding stresses will be relieved. The assembly fish plates are then removed, the welder's helper holds the head bars in place while the welder "tacks" them on, and then they are welded full length in the usual way. The first few joints welded had double beads, but as this was found unnecessary all later joints made in track have had single beads.

Testing for electrical conductivity on hundreds of base-plate joints shows that the conductivity of the joint is greater than the same length of rail in the ratio of about 3.00 to 2.75.

The cost of a base-plate joint on 7-in., 91-lb. rail compared to the fish-plate joint is given in Table 2.

TABLE 2.—COMPARATIVE COSTS

Base-plate Joint	
Tie plates (none).....	\$0.00
Two 1 by 1½ by 16-in. machine steel, 13 lb., and one ¾ by 7 by 28-in. machine steel, 42 lb.—55 lb. @ 2.1 cts.....	1.16
Three pounds welding steel @ 8 cts.....	0.24
Labor placing and removing assembly plates.....	0.50
Labor welding: 1⅓ hr. welder @ 65 cts. and 1⅓ hr. helper @ 40 cts.	1.40
Total cost base-plate joint.....	\$3.30
Fish-plate Joint	
Two tie plates.....	\$0.30
Two ⅞ by 26-in. specially rolled welding plates, 62 lb.	2.41
Three pounds welding steel.....	0.24
Labor assembling.....	0.40
Labor welding.....	1.40
Total cost fish-plate joint.....	\$4.75
For 9-in. section 104-401 girder rail	
Base plate joint, same as for 7-in., 91 lb. rail.....	\$3.30
For Fish-plate Joint	
Two tie plates.....	\$0.30
Two 1 by 26-in. plates.....	3.41
Three pounds welding steel.....	0.24
Labor assembling.....	0.40
Labor welding.....	1.40
Total fish-plate joint.....	\$5.75

The saving made with the base-plate joints on light sections of T-rail is not very large, but the base-plate joint is so much stronger than the fish-plate joint on the light T-rails that it makes the base-plate type a distinct advance in the development of welded rail joints.

Other Studies and Various Applications.—It is possible that later experience will show an economy in having a rolled plate with thin ends and thick center, as in Fig. 26. Although this would defeat the scheme of avoiding specially rolled material, it is presented as a thought for further investigation. It may develop that shorter and thinner base plates can be used and

still provide necessary strength. Likewise, a different length and shape of the head bars may be found desirable.¹

I have made interesting developments in the way of designing and building compromise joints (*i.e.*, joints between rails of



FIG. 24.—A 7-in. T-rail laid and welded with base-plate joints.



FIG. 25.—A 7-in. T-rail laid and welded with base-plate joints.

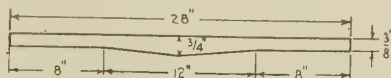


FIG. 26.—Rolled plate with thin ends and thick center.

different heights, such as 7 to 9 in.) by placing several plates under the low rail to make rail heads the same height and a long base plate under the entire joint. A forged compromise

¹ Recent tensile tests at the Bureau of Standards on base-plate joints indicate the advisability of using head bars of the same length as the base plates.

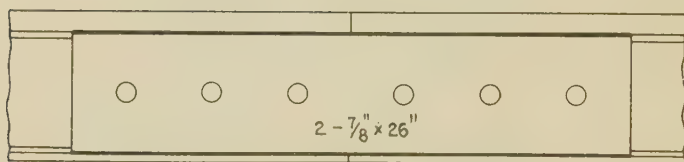
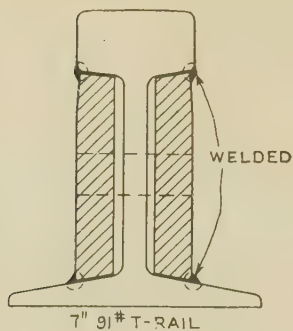


FIG. 27.—Fish-plate type arc-welded rail joint.

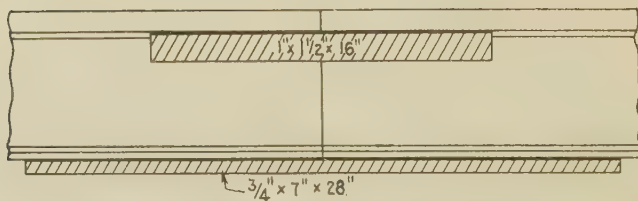
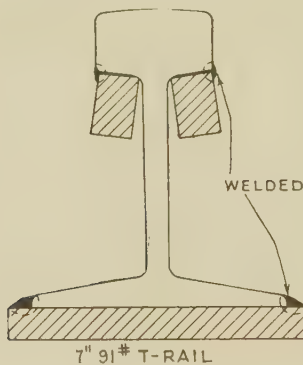


FIG. 28.—Base-plate type arc-welded rail joint.

joint costs \$16 to \$20 in place, whereas a base-plate compromise joint costs \$5 to \$6 in place, and can be made on the job with stock plates.

Summary.—The base-plate joint is a new and distinct improvement on the present fish-plate joint and has the following advantages:

1. Stronger.
 2. Cheaper by \$1.45 to \$2.45 on 7-in. or 9-in. rail.
 3. Permits use of the same size of plates and bars for several sections of rails, thereby reducing amount of stock on hand.
 4. Plates and bars can be purchased in open market and do not have to be specially rolled or punched.
 5. Use of 28-in. base plates obviates necessity of using two tie plates on joint ties.
 6. Makes a strong and cheap compromise joint.
- 

PART V

STABLE ARC WELDING ON LONG DISTANCE PIPE
LINES

BY

B. K. SMITH

CHAPTER I

METHOD ON HIGH-PRESSURE LINES

For several years past some of the pipe-line people have employed the oxyacetylene process with successful results. The pipe-line engineers have considered this a radical change from the old-time method of mechanical couplings and fittings. This change was not made on account of economy, but because of quality. With the threaded joints the pipe is weakened on its threaded ends. With the welded joints the pipe is reinforced on the welded ends. From experience in the field I have found that some constructed pipe lines with fittings become defective or aged first at the threaded joints while the pipe itself is still good for many years of service.

At the beginning of the welding era we welded both ends of the threaded coupling, thus making it cost twice as much. Later on we decided to order the pipe with no threads, and the cost was reduced, amounting to about the same cost as the fittings. The outstanding features in favor of the welding were as follows:

1. It cost no more than the fitting.
2. Had no maintenance expense.
3. Did not corrode at joints.
4. Had less weight.
5. Required less room.
6. Pipe line was better insulated.
7. It had a uniform interior diameter.

For these reasons its general use is predicted in the future. Newspapers, trade papers, pipe-line engineers, welding supervisors, contractors, and manufacturers of oxyacetylene and electric welding equipment are all praising the oxyacetylene-welded pipe joint. However, I have found that electric welding would do all of that as well, and will return to the owners a bonus for dividend purposes from 35 to 50 per cent of the cost of the welding construction. I have studied and experimented with the stable-arc process for welding long-distance pipe lines for years, and this article is to show that arc welding is the logical

process to use because it is faster and cheaper without lowering the quality of the weld.

Stable-arc welding on gas or oil pipe lines is one of the newest methods and at the same time one of the most far reaching in its effect on modern civilization.

The successful application of electrical energy converted into heat by the arc has made possible economical mass production of a wide variety for joining pipes that formerly was unattainable. Working hand in hand with testing and pipe-line engineers, I have achieved a correct and successful application of electric welding heat to pipe lines.

No application in my opinion has received greater benefit from the advanced progress in stable-arc welding than the joining of pipe lines. The pipe-line industry is about half a century old, and today these arteries of the oil industry are over 80,000 miles long. Through swamps, forests, hills, and under rivers they carry 2,000,000 barrels of crude oil daily. Another 170,000 miles of pipe lines convey many billions of cubic feet of natural or artificial gas.

From the very beginning the joining of these pipe lines was made with heavy cast-iron and malleable-iron fittings or by some special patent mechanical joint which depended upon being leak proof from a rubber gasket. A small percentage of this mileage was constructed by welding the joints with the oxyacetylene process, and this has furnished a far more superior joint than the cast-iron or malleable iron fitting. Today about 25 per cent of oxygen and acetylene gases manufactured in the Southwest are utilized for welding pipe lines.

The stable-arc-welded joint is also superior in quality compared with the mechanical coupling. It requires no future maintenance and will last in the ditch as long as the pipe, but the cost of welding this joint is about one-half that of the oxyacetylene welded joint.

In addition to welding of pipe lines, the stable-arc-welding process will open an enormous field in the manufacture of large fittings from fabricated pipe. It will help the plumbing shops to get the work done faster and cheaper and save delays. It will also cost less in constructing water-pipe lines, refrigerating tubes, steam-power lines, and compressed-air lines.

It is hoped that the efforts and the success connected with this first electrically welded pipe line will benefit in reducing the cost

of construction of such pipe lines. We might expect that such reduction will be reflected in the price of gasoline, oil, water, and refrigeration, and possibly the plumbing bill will be on a downward scale.

The author has been so convinced of the practicability of the welding method that he considered that the best way to advance the knowledge of it would be to secure a contract and do the job himself.

The Advance Bag & Paper Company of Boston, Mass., was building a large paper mill at Hodge, La., and as it needed natural gas for its plant, it sent out bids through its agent, the Jordan Drilling Company of Monroe, La., for welding a 7-in. high-pressure gas line 45 miles long, from Lamkin to Hodge, La. The author secured the contract, with the provision that before he commenced actual welding some severe tests should be made.

These test welds were made at the Hodge-Hunt Lumber Company shops. Five lengths of pipe each 30 ft. long were laid on 1-ft. high skids and welded for these tests. They were then tested to 1,250-lb. hydrostatic pressure without finding any leaks. Then a heavy hammer was used on each weld while under the 1,250-lb. pressure. Not satisfied with this test, all the center skids were removed, leaving the five lengths of pipe hanging on the two ends, and then the superintendent jumped up and down on the middle of the pipe while under the 1,250-lb. pressure, and no joint broke or leaked.

On Aug. 25, 1927, we started on the 45 miles of pipe welding. We found that the lay of the country was unfavorable, as the line passed through a forest full of ravines, hills, and ditches. Every 50 ft. or less we had to go up a hill or around a bend, which necessitated many more bell joints.

We started with three Lincoln stable-arc gas-driven sets. One was driven by a Buda engine and the other two were driven by Waukesha engines. Each machine was mounted on an ordinary wagon drawn by two mules. However, the development of a lighter machine would be better for this class of work, such as a duplex machine consisting of two 200-amp. generators driven by one gasoline engine in the center, provided such duplex machine would not weigh over 3,000 lb. I also developed an easier and cheaper carrier by riding these machines on the pipe, as shown in Fig. 1.

After several tests with various kinds of electrodes, we found a coated electrode that was entirely satisfactory and that produced a reasonably ductile and neat weld. This wire did not splash or explode so much, which eliminated pinholes in the weld and the burning of holes on the operator's clothes.

Among the provisions under which the author's company, the Big Three Welding & Equipment Company, of Houston, Tex., took the contract, it agreed to furnish all the necessary welding equipment known as the electric-arc process, supplies such as welding rods, and the necessary operators including supervision. The price agreed upon was \$1.25 per joint. The pipe was to

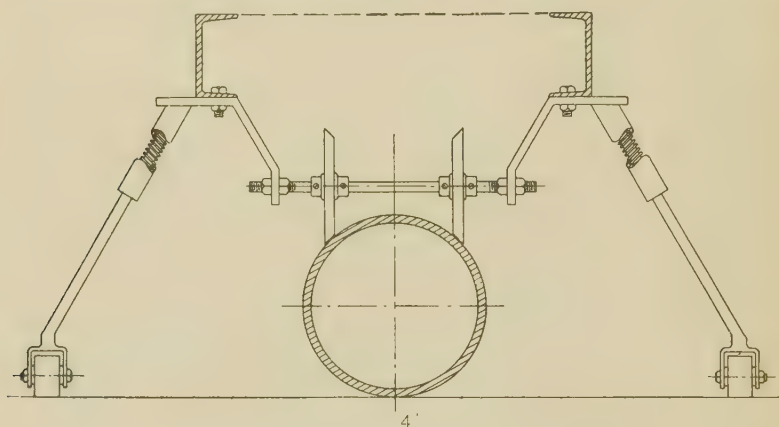


FIG. 1.—Machine rigged to ride on the pipe.

stand a test of 600 lb. pressure. The work was to be completed within 90 days. The pipe was to have a carbon content not higher than 0.35 per cent.

On Sept. 1, I promised my operators a bonus of \$100 if they could weld the first 5 miles of pipe without finding a single leak. From past experiences I have found that the average of leaks on pipe lines is 5 per cent, regardless of process. However, on Sept. 9 we finished the first 5 miles and tested with 650 lb. hydrostatic pressure, and no leaks or pinholes were found in about 1,100 joints. We then placed one more machine on the job and welded half a mile per day.

Eliminating Cracks and Pinholes.—All operators were examined before we started by making welds on ordinary boiler plate. Each weld on the pipe was made in two layers. The

first layer took care of about one-half the thickness of the material on the pipe, and then we welded the other half, leaving a reinforcement of about $\frac{1}{8}$ in. above the surface of the pipe. This was to control the elimination of pinholes. If an invisible pinhole develops on the first bead or weld, the top or second weld will cover it. If the second weld has any pinholes, the first weld underneath will cover it. An inspector on the job scrutinized every welded joint with a magnifying glass, and he would not pass a joint if a pinhole or crack was visible. ✓

Method of Welding.—We had four operators on the job who had the same speed in welding. For instance, we started on five lengths of pipe to weld four joints, and they had one helper who turned the five lengths of pipe. Each one started to weld at his respective joint from the bottom up. After welding 7 in. they were ready to turn the pipe about 7 in., and the one helper served all four welders by turning the pipe. Again, all four operators welded from the bottom up about 7 in., and again all finished at the same time. For the second time the helper, with the aid of a chain tong pipe wrench, turned the pipe about 7 in., and the operators again started from the bottom up, and thus all completed the weld at the same time. When the second top weld was made, the end of the weld was tied in or continued for about $\frac{1}{2}$ in. on the side of the joint. In doing so we eliminated a possible blowhole or a low spot on the welding line.

That was the method employed throughout the job, and we found that while this was very simple, it eliminated guesswork and performed the arc welding in such a way that we could control its efficiency. By having four men who could make the welds at a uniform speed, we saved the cost of three helpers. However, a greater saving could have been obtained if more machines could have been used on the job.

For the bell joint operator we had a separate Lincoln gas-driven welder, and he made his welds alone while the pipe was stationary, and as it could not be turned, he did not need a helper. His work was to join the five-length sections that previously were welded by the crew of four welders.

Method of Testing.—The first 5 miles we welded from the Hodge end. We used a hydrostatic test, then we moved our equipment to the Lamkin end, where the gas wells are. The reason for doing this was that we had over 1,000 lb. gas pressure at the well and could make tests a great deal cheaper than with

✓ the hydrostatic method. From then on each 5-mile section was tested by hammering the welds and inspecting with soap-suds while under 600 or 650 lb. pressure. Each joint was stamped with the welder's number, this giving an effective check on the work of the operators.

CHAPTER II

COSTS AND SAVINGS

Each man welded one 7-in. joint in 15 min., but with some intermittent loss of time each operator produced three welded joints in 1 hr. The welding machines used about 1 gal. of gasoline per hour at a cost of $11\frac{1}{2}$ cts. per gallon, and 6 qt. of lubricating oil for every 55 hr. of actual welding. We had one helper that kept the machines supplied with gasoline and oil and who provided electrodes for the operators. The crew represented five electric welders at \$1 per hour, two helpers at 25 cts. per hour, one night watchman at 25 cts. per hour, and one pipe cleaner at 30 cts. per hour, his work being to clean the pipe of the heavy coat of asphaltum applied at the mill. This asphaltum retards the welding with any process, and particularly with the electric-arc process. We first tried to burn it off, but this proved too costly; so we found a duco paint remover, and one helper with a paint brush cleaned the joint in less than a minute. That solved one of the worst troubles we had.

The cost per joint on welding a 7-in. pipe was as follows:

Labor.....	\$0.34
Interest on investment.....	0.02
Depreciation.....	0.01
Gasoline.....	0.03
Lubricating oil.....	0.01
Welding rods.....	0.03
Overhead.....	0.10
Transportation of men.....	0.01
Freight on machines.....	0.03
Total.....	\$0.58

The lowest bid on this pipe line that the owners received from various oxyacetylene companies was \$2.75 per joint. My company's bid, and the only one with the electric arc, was \$1.25 per joint. Thus we saved the owners with our method \$17,500 on approximately 11,000 joints.

With the oxyacetylene method the cost of labor, overhead, and welding rods is all the same. The interest might be a fraction lower, but the depreciation is higher because of losing and burning tips, of defective regulators and gages owing to rough transportation, the burning of hose, etc.

With the stable-arc welding, as compared with the oxyacetylene, the cost can be cut in half. Presuming that some other companies were not so well organized as we are, the cost with electric welding at the beginning would be at least one-third lower than with the oxyacetylene.

Fifty-five hours of actual welding required 55 gal. of gasoline and 6 qt. of lubricating oil, at a total cost of \$6.70, or a little over 12 cts. per hour. If the same job had been done with the oxyacetylene process the cost per joint would have been as follows:

Labor.....	\$0.34
Interest on investment.....	0.01
Depreciation.....	0.02
20 ft. oxygen @ 1 ct. per foot.....	0.20
18 ft. generated acetylene @ 1½ cts.....	0.27
Welding rods.....	0.04
Overhead.....	0.10
Transportation of men.....	0.01
Cost of clerk to check tank numbers.....	0.03
Freight on oxygen tanks from charging plant to depot and return.....	0.10
Handling of tanks from depot to right of way and return to depot.....	0.02
Total.....	<u>\$1.14</u>

We arc-welded the first 30 miles of pipe without finding any cracks or leaks, and most of the pipes were thrown in the ditch and covered. In order to find the real speed of arc welding, I made a proposition to the welders to pay them on a piecework basis of 50 cts. per joint, with the guarantee that they would do the work on their own time if the quality should be jeopardized for the sake of speed. I watched the results of the next 5 miles and found that each welder made from 30 to 40 joints per 8-hr. day. The owner's inspector approved every weld, and upon the next test of 650 lb. pressure we found no leaks on the 5-mile section. Therefore I consider that better speed can be made

with the electric-stable-arc process than with any other autogenous welding process known.

In comparing the foregoing tables of cost, it appears that if the oxygen and acetylene were furnished free of charge the cost of freight alone on these tanks would be higher than the cost of arc welding.

Presuming that we would weld at least 11,000 joints, our total cost with the stable arc would be \$6,380. If the same amount of joints were done with the oxyacetylene process, the cost would be \$12,540. If, however, compressed acetylene and tanks were used, the cost would increase 18 cts. per joint for acetylene and 10 cts. per joint on freight, a total of 28 cts., or a total for the four joints of \$1.42. In that case the total cost of 11,000 joints would be \$15,620. ✓

This reduction of cost will benefit the following industries:

Public utility or gas companies that now are facing a daily crisis; the one demanding increased production and decreased costs. Through this critical situation they expect from us developments of new methods on pipe lines for the saving of time and money.

Oil companies will benefit by this method because they manufacture the very product that is used for portable stable-arc welding. Instead of purchasing another company's product, they can use their own product and also stimulate new sales for gasoline and oil to other companies who may construct new pipe lines.

Pipe-fitting manufacturers, who can make various sizes and shapes of fittings from fabricated pipe at a much lower cost than the present one for heavy cast-iron or malleable-iron fittings.

Plumbing and steam fitters, who also can make their own fittings on the job, such as T's, elbows, and crosspieces. They can weld pipes together and save a great deal of time, labor, and material.

On the whole job we did not have an acetylene welding or cutting torch for any purpose. In some few instances where we had to cut a hole in the pipe, we used the carbon arc, and although somewhat slower, we made it up from the time of getting the gases and adjusting the regulators.

On Dec. 15 this pipe-line job was finished. In order to illustrate the work on this pipe line, some photographs were taken of the work.

Figure 2 shows an actual stable-arc weld which can be performed with the Lincoln arc-welding machine. Notice the ripples and the wavy marks of this weld. This is done by the semicircular motion of the arc which makes it appear like a machine weld. Some gas competitors would not believe that this was an electric-arc weld and thought it an acetylene weld, yet it is a stable-arc weld, and it not only is neat, but solid and ductile.

Figure 3 shows an arc-welded joint finished. Notice the ground plate. It was made of a piece of 8-in. pipe cut in half



FIG. 2.—The weld and the machine.

so that it would fit a 7-in. pipe line. Notice also the Lincoln stable-arc welder coupled to a Buda engine mounted on a wagon drawn by two mules. The pipe lies on skids and is thrown in the ditch after 5 miles is welded, tested, and accepted.

Figure 4 shows the bell-hole man. His duty is to weld five-length sections where the pipe cannot be turned. His welds are just as neat in appearance and as strong as the ones welded by the four welders. Notice a small electrode rack lying on the ground at the side of the operator. Each pipe-line welder has one of these racks in which to carry the electrodes or welding wire. This rack is made of sheet steel with a carbon bottom. It has a wooden block with holes drilled inside as a core. This wooden

block is set inside the sheet-steel box and rests on the carbon plate. The electrodes are placed in the holes of the wooden



FIG. 3.—An arc-welded joint as finished.



FIG. 4.—Joining five-length sections.

block, and the whole box is grounded by a branch cable from the main ground. This rack was made for the purpose of saving the

pieces of electrodes when they became too short to handle in the holder. All the welder has to do is to touch the short pieces



FIG. 5. - Pipe ready to be dropped to ditch.

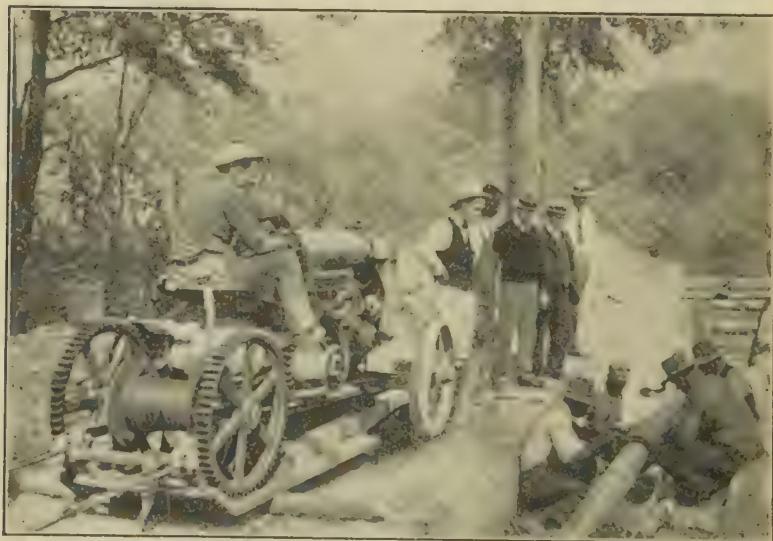


FIG. 6.—Weighting line to stay on river bottom.

into any one of the electrodes placed in the rack, and he gets a perfect joint with the next electrode. This saves us time in opening the holder, throwing the short pieces away, and placing

another electrode in place. It also saved on the entire line about 600 lb. of electrodes. Notice also the Lincoln stable-arc welder which is coupled to a LeRoy engine; also the tool boxes on each side, so that the welder can help himself in emergencies. On the rear end he has also a cable reel on which to wind his cable. On the rear end of the wagon he has a 55-gal. barrel of gasoline and a $1\frac{1}{2}$ -gal. can of lubricating oil.

Figure 5 shows the woody districts along most of the line. Hills occurred frequently, some of them dropping down 60° . This section of pipe is ready to be thrown in the ditch.

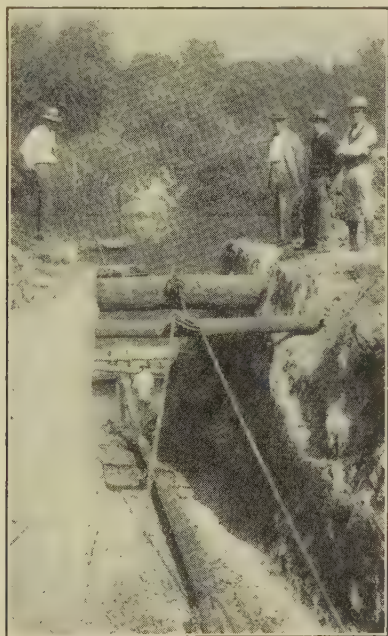


FIG. 7.—Pipe lowered in trench to river.

Figure 6 shows the crossing of the Ouachita River. The banks were about 60 ft. high, and the pipe was bent so as to reach the bottom of the river, and then the sections were arc-welded one at a time and strung across pontoons. The four workmen are bolting on some heavy cast-iron weights so as to keep the pipe on the bottom of the river. Two sections of 6-in. pipe were strung across. During this operation one of the ropes that held two of the pontoons broke, and the swift current

carried the pipe about 60 ft. It was feared that every joint would be broken, but after the pipe was drawn back in place with the help of a tractor, it was found that every joint was stout.

Figure 7 shows the pipe being lowered into the water.

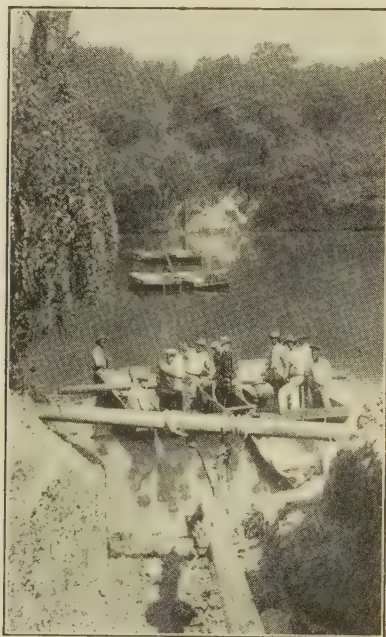


FIG. 8.—Watching for leaks from pipe in river.

Figure 8 shows the second connection made on the Ouachita River crossing, the pipe already having been lowered on the river bed. A gas pressure of 650 lb. was put on the line, and the men watched for leaks which would show as bubbles in the water, but no leaks were found.

PART VI

ARC WELDING AS APPLIED TO CONSTRUCTIVE WORK
AT THE PHILADELPHIA NAVY YARD

BY

M. M. KENNEDY and F. H. WIELAND

INTRODUCTION

Previous to the construction of the U.S.S. "Henderson" at the Philadelphia Navy Yard in 1915, welding of the ferrous metals was accomplished almost entirely by the forge method. Gas welding was employed only to a minor degree, and arc welding had not been introduced.

Experimental gas welds of steel shapes, forming staples, and other watertight and strength members for the hull structure of the U.S.S. "Henderson" proved satisfactory, and as a result forge welding of steel shapes was abandoned entirely and gas welding adopted.

During 1918, the first arc-welding equipment was installed for production work, and after a short interval this method superseded gas welding on all hull structural work, arc welding proving to be more economical, entirely satisfactory with regard to strength, and possessing the added advantage of greater mobility in use and application.

The increasing use of arc welding in the fabrication and erection of structural members and hull fittings in place of the riveting and calking method is illustrated in the two projects described.

CHAPTER I

BOILER ROOM FOR TEST PURPOSES BUILT FOR FUEL-OIL TESTING PLANT, NAVY YARD, PHILADELPHIA, PA.

In connection with the recent naval building program the Navy Department authorized exhaustive tests conducted on the high-pressure boilers it purposed to use in the new 10,000-ton cruisers. To conduct these tests it was necessary to construct a practically airtight building of sufficient strength to withstand an internal air pressure of 0.54 lb. per square inch, which is approximately 78 lb. per square foot of surface. The plans for this building, as prepared, called for construction by arc-welding methods. ✓

The building is rectangular in form, with internal dimensions as follows: length, 45 ft., width, 30 ft., height, 25 ft. One end of the building is portable so as to permit the boiler to be tested being placed in the building as a complete assembly.

The side walls and roof are constructed of 10 lb. per square foot steel plate. The main framing or stiffeners, all of which are located externally and run vertically from foundations to roof, are I-beams 3 by 12 by 5 in., 31.5 lb. weight. The intercostal stiffeners are channels $3\frac{1}{2}$ by 6 by $3\frac{1}{2}$ in., 15 lb. weight, spaced at intervals between the main framing. The entire building with the exception of the portable end is anchored to a concrete foundation.

Welding Details.—The design was such that the plates for the side walls and roof overlapped $1\frac{3}{4}$ in. on each flange of the I-beam framing. The toes of the I-beams were welded continuously to the plate, and the lap edges of the plates were tack welded to the I-beams with welds $\frac{5}{8}$ -in. long at 5-in. intervals. The plates were continuous from the foundation to the roof for the sides. Owing to the larger dimensions of the roof, it was necessary to use pieces of plate butted together. These butts were welded on the center of the intercostal channels.

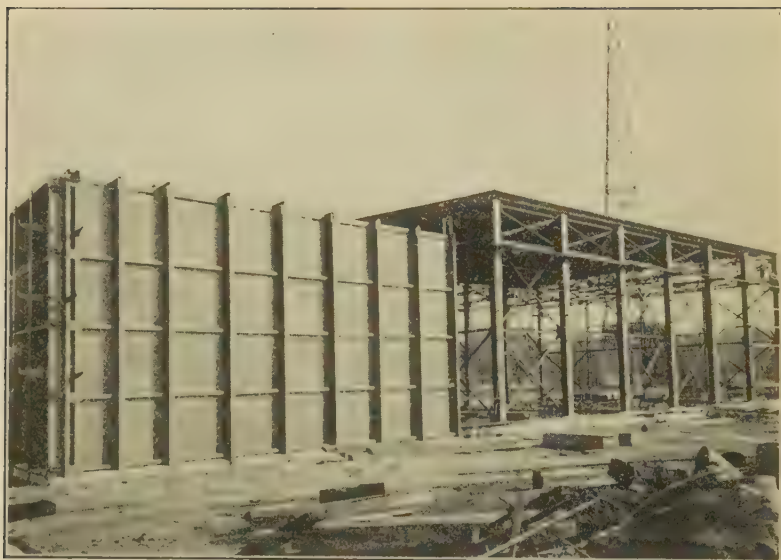


FIG. 1.—Boiler test room, Navy Yard, Philadelphia, Pa.

View from northwest showing west wall and north end, the latter being a single portable panel.

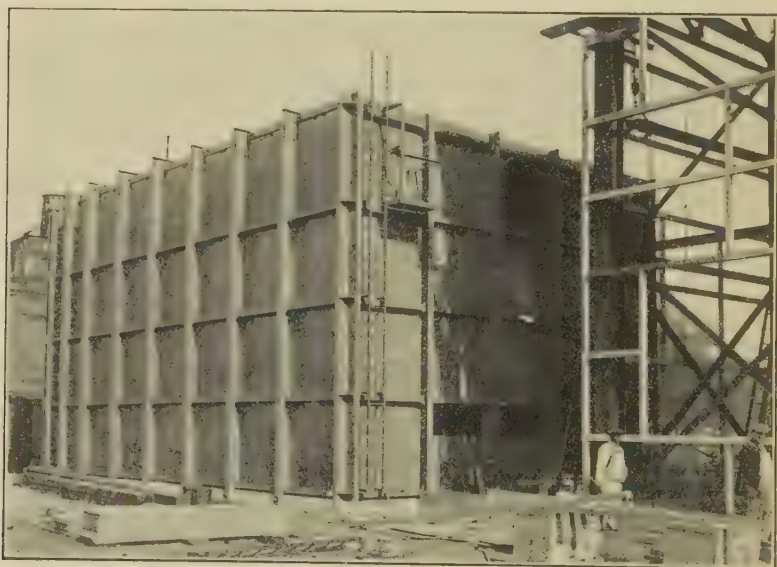


FIG. 2.—Boiler test room, Navy Yard, Philadelphia, Pa.

View from southwest showing west wall and south end. The rectangular openings are fitted with heavy glass for observation purposes. Access door to air-lock chamber is on extreme right. Circular opening is for air-supply connection. Ladder, platform, and door are for access to internal air duct.

The horizontal intercostal stiffeners on the sides were placed heel side up and the heel welded continuously to the plate to prevent the entrance of moisture, with subsequent corrosion.



FIG. 3.—Boiler test room, Navy Yard, Philadelphia, Pa.

Interior view looking south, showing air-lock chamber for access, observation windows, and overhead air-distributing duct.

and tack welded at 3-in. intervals along the toe. Similar members on the roof were continuously welded.

Corner angles were not fitted at the junction of the sides and roof, but instead the plates were fitted, and a heavy internal fillet weld completed the connection. Similar methods were

employed in securing the air ducts, air lock, etc., to the interior of the building.

It will be seen from this description and the accompanying photographs of the completed building (Figs. 1, 2, 3, and 4)



FIG. 4.—Boiler test room, Navy Yard, Philadelphia, Pa.

Looking north, showing boiler in position; notice smooth finish of side walls and roof; opening in roof is for uptake and stack.

that bolts or rivets were used only where it was considered desirable for assembly and erection purposes; *i.e.*, at the ends of the intercostal stiffener brackets, at the junction of sides and ends, and at the junction of side and roof stiffeners.

Erection.—In order to minimize the field work to the greatest possible extent, the sides were shop-assembled and welded as complete individual units, while the roof was shop-assembled, welded, and shipped in four sections on account of erection conditions.

The total weight of the building approximated 50 tons, divided as follows: Side, 10 tons; end walls, $7\frac{1}{2}$ tons; roof, $12\frac{1}{2}$ tons; air ducts, air lock, etc., $2\frac{1}{2}$ tons.

The shop work was done in the structural shop, which is approximately $\frac{1}{3}$ mile from the place of erection. Transportation and erection were accomplished with two locomotive cranes.

Conclusion.—Considering the erection of a similar building by riveting and calking methods, it is conservatively estimated that the arc-welding method pursued in this construction resulted in an actual saving of 40 per cent in labor and 100 per cent in time. ✓

CHAPTER II

EXPERIMENTAL ARC WELDING OF STEEL T-BARS TO ARMOR PLATE

(To Establish Their Strength Values in Connection with Arc
Welding of Watertight Blister to the Armor Plate of the
U.S.S. "Arkansas")

General.—In order to provide additional protection against torpedo attack, the plans for the modernization of the U.S.S. "Arkansas" called for the attachment of a watertight blister over the present armor plate for a certain distance above and below the waterline. The construction of a blister attached to the hardened or outside surface of the armor plate was a new problem in hull construction. Previous experience in securing an experimental blister to the armor plate of the U.S.S. "South Carolina" by arc welding indicated that there were possibilities to be realized by this method, and it was for this purpose that the present experiment was conducted.

Description of Blister and Attachment.—The blister to be attached to the armor plate of the U.S.S. "Arkansas" was to extend from frame No. 15 to frame No. 126, as shown in Fig. 5, on the port and starboard sides of the ship. The blister started at practically no thickness, gradually increased in thickness, and then tapered off again toward the stern. The blister was to be constructed of plate varying in weight from 10 to 25 lb., and its members were to be attached to the armor plate in vertical planes throughout the length.

Conditions in Selecting Shape to Be Welded.—Consideration of the conditions existing when a torpedo or other weapon, such as a mine, exploded against the blister, led to the choice of a T-bar for the member that was to be welded to the armor plate. The original explosion, with its impact, may occur directly over one frame, in which case that particular section would be in direct compression; but since it would probably buckle, the frames on each side, instead of being in direct compression, would

be loaded eccentrically and the member actually welded to the armor be subjected to quite a heavy fore-and-aft thrust.

The use of a T-bar to be welded to the hardened armor face possessed the following advantages over an angle similarly welded: (a) It enlarged the lever arm of the welds against the side thrust, and (b) it made the construction equal in strength and flexibility in both directions. The former is quite a desirable factor when consideration is given to the spalling of the armor.

Selection of Method for Arc-welding T-bar to Armor.—

Previous experience had demonstrated conclusively that it was impossible to weld satisfactorily structural steel shapes directly to the hardened face of armor, as the difference in expansion between the armor plate and the weld metal caused the weld to crack, but that it could be accomplished by arc-welding a bead to the armor and then arc-welding the shape to the bead. This method was therefore used in welding the T-bar to the armor.

In depositing the bead to the armor a strip of copper was used about $\frac{1}{8}$ in. thick and $\frac{3}{16}$ in. greater in width than the width of the T-bar to be attached. The copper strip was

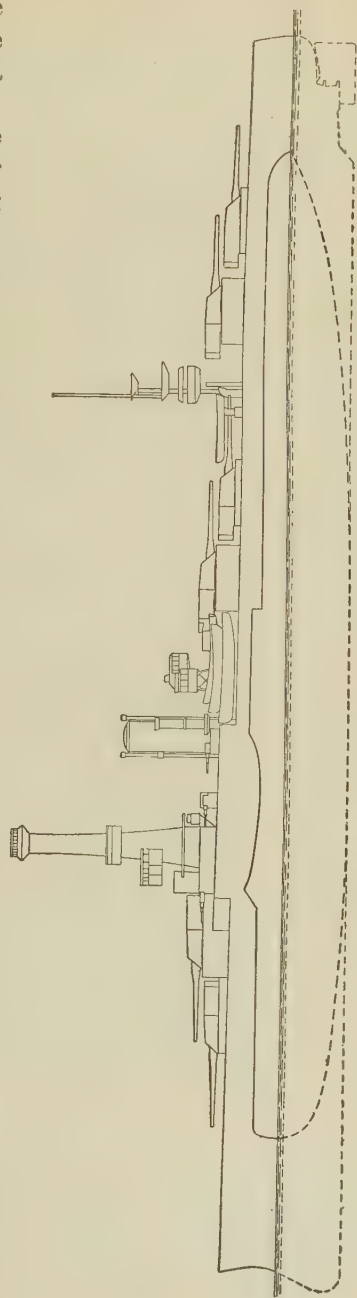


Fig. 5.—Plan of U.S.S. "Arkansas" showing blister as welded on each side.

laid along the armor in the location where the T-bar was to be attached, and a bead was laid on each side by arc welding. The copper strip was then removed and the shape substituted and arc welded to the bead. The copper strip was used because it was



FIG. 6.—Armor plate after completion of welding.

unaffected by the arc-welding process, and on removal left a bead of welded material in which there were no irregularities, thus facilitating in the location of the T-bar.

It was also known from previous experience that arc welding on hardened armor plate caused spalling of the plate. It was

believed that spalling could be partially eliminated by intermittent welding rather than by continuous welding. ✓

General Outlines of Test.—While the foregoing considerations were believed correct, it was desired to prove by actual experiment the strength of 3- by 4½- by ½-in. by 8.4-lb. T-bar, arc-welded to armor plate. The effect of different welding conditions (*e.g.*, amperes flowing in the arc, speed of welding, continuous welding, intermittent welding) was to be determined. To represent each condition of arc welding, five tension specimens 4¾ in. long and one bend specimen 2 ft. long were arc-welded to a piece of scrap armor plate 6 by 4 ft. by 10 in. thick. The arc welding was done on the armor plate while it stood in a vertical position, in an unheated shop, so as to simulate actual conditions under which the production work would be done on the side of the ship as she lay in the drydock during the winter months. Figure 6 shows the armor plate after completion of the welding. The electrodes used were 5/32 in. in diameter by 14 in. long and were obtained under regular navy contract and taken from one delivery so as to be as nearly uniform as possible.

The nameplate data of the Lincoln machine used is as follows:

Lincoln Electric Co., Cleveland, Ohio

Motor.—No. a-5935. Frame F.K.W. List 12844. Volts 220. No-load r.p.m. 1800. Amperes 40. Rating 50°. Generator.—No. a-5935. Frame F.K.W. List 12844. Volts 50/0. No-load r.p.m. 1800. Amperes 200. Rating 50°.

Two types of test specimens were used because it was desired to determine the value of the structure both in tension and bending. The tension tests were made by exerting a pull by means of hydraulic jacks on each section of the T-bar through a ring which had been calibrated for deflection against load. The ring and gear used for this test is shown in Fig. 7.

The bending tests were made by laying down the vertical member of the T-bar by successive blows with a 16-lb. sledge hammer until it was in contact with the flange. ✓

Results of Test.—The results of the test are tabulated in Tables 1 to 6, together with photographs of the respective specimens, before and after test to destruction (Figs. 8 to 18).

It will be noticed that under all conditions of welding, cracks appear in the armor just outside the bead. There are also

similar cracks near the inside edge of the bead which are hidden by the T-bar. These cracks are best illustrated in test No. 3 (Table 3) after removal of the T-bars. This extreme condition

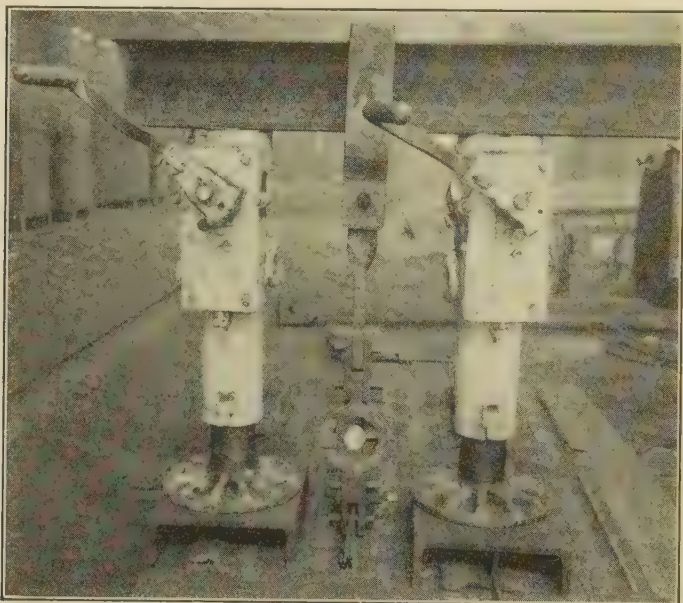


FIG. 7.—Calibrated ring and hydraulic jacks set up to make tension test.

is considered to be the result of improper welding methods. The presence of the cracks had no apparent effect on the properties of the assembly either in tension or bend.

TABLE 1.—TEST NO. 1, OBSERVATIONS AND RESULTS
Desired Conditions

	Welding bead to armor	Welding T-bars to bead
Arc amperes.....	165-170	150-155
Speed of deposition.....	2 ft. per hour	2 ft. per hour
Method of welding.....	Deposit one electrode and skip 1 ft. before making next deposit	

Observations
Welding Bend Test Specimens

Electrical (of arc) as measured at machine.....	165 amp. 19 volts	150 amp. 18 volts
Time required.....	2 hr., 30 min.	2 hr., 20 min.
Length of weld.....	49 in.	48 in.

Welding Tension Test Specimens

Electrical (of arc) as measured at machine.....	165 amp. 20 volts	150 amp. 19 volts
Time required.....	1 hr., 30 min.	2 hr., 5 min.
Length of weld.....	54 in.	44 in.

Results

Bend Test

Vertical member of T-bar laid flat against flange; assembly showing no signs of failure.

Tension Test

Specimen number	Maximum load, pounds	Distortion of T after failure, inches	Remarks
1	10,750	$\frac{1}{4}$	Broke through both welds
2	9,000	$\frac{1}{4}$	Broke through weld on one side
3	12,250	$\frac{1}{4}$	Broke through weld on one side
4	13,000	$\frac{1}{4}$	Broke through weld on one side
5	10,000	$\frac{3}{8}$ ₂	Broke through weld on one side, and pulled weld from armor on other side
Average.....	11,000		

TABLE 2.—TEST NO. 2, OBSERVATIONS AND RESULTS
Desired Conditions

	Welding bead to armor	Welding T-bars to bead
Arc amperes.....	165-170	150-155
Speed of deposition.....	2 ft. per hour	2 ft. per hour
Method of welding.....	Continuous	Continuous

Observations

Welding Bend Test Specimens

Electrical (of arc) as measured at machine.....	165 amp., 19 volts	150 amp., 19 volts
Time required	1 hr., 40 min.	2 hr., 20 min.
Length of weld.....	52 in.	48 in.

Welding Tension Test Specimens

Electrical (of arc) as measured at machine.....	165 amp., 19 volts	150 amp., 19 volts
Time required.....	2 hr., 5 min.	2 hr., 5 min.
Length of weld.....	57 in.	44 in.

Results

Bend Test

Vertical member of T-bar laid flat against flange; assembly showing no signs of failure. After completion of bending test a crack was noticed in the weld joining T-bar to the bead. This crack seemed to have no effect upon the assembly in so far as the bending test was concerned.

Tension Test

Specimen number	Maximum load, pounds	Distortion of T after failure, inches	Remarks
1	9,250	$\frac{1}{4}$	Broke through weld on one side
2	12,500	$\frac{1}{4}$	Broke through weld on one side
3	8,750	$\frac{1}{8}$	Broke through weld on one side, and pulled away from armor on other side
4	7,000	$\frac{1}{16}$	Broke through weld on one side, and pulled away from armor on other side
5	10,250	$\frac{1}{16}$	Failure by partially tearing of bead from armor and partially by weld tearing from bead
Average.....	9,550		

TABLE 3.—TEST NO. 3, OBSERVATIONS AND RESULTS

Desired Conditions

	Welding bead to armor	Welding T-bars to bead
Arc amperes.....	165-170	150-155
Speed of deposition.....	As rapid as possible	As rapid as possible
Method of welding.....	Deposit one electrode and skip 1 ft. before making next deposit	

Observations

Welding Bend Test Specimens

Electrical (of arc) as measured at machine.....	165, amp., 20 volts	150 amp., 19 volts
Time required.....	52 min.	1 hr., 30 min.
Length of weld.....	52 in.	48 in.

Welding Tension Test Specimens

Electrical (of arc) as measured at machine.....	165 amp., 19 volts	150 amp., 19 volts
Time required.....	1 hr., 11 min.	1 hr., 27 min.
Length of weld.....	54 in.	44 in.

Results

Bend Test

Vertical member of T-bar laid flat against flange; assembly showing no signs of failure.

Tension Test

Specimen number	Maximum load, pounds	Distortion of T after failure, inches	Remarks
1	12,000	$\frac{1}{32}$	Pulled off armor on one side
2	11,500	$\frac{1}{8}$	Broke in weld on one side
3	10,000	$\frac{3}{32}$	Broke in weld on one side, and tore away from armor on other
4	5,750	0	tore away from armor on both sides
5	9,000	0	Tore away from armor on one side
Average.....	9,650		

TABLE 4.—TEST NO. 4, OBSERVATIONS AND RESULTS
Desired Conditions

	Welding bead to armor	Welding T-bars to bead
Arc amperes.....	190-200	165-170
Speed of deposition.....	2 ft. per hour	2 ft. per hour
Method of welding.....	Deposit one electrode and skip 1 ft. before making next deposit	

Observations

Welding Bend Test Specimens

Electrical (of arc) as measured at machine.....	195 amp., 20 volts	165 amp., 20 volts
Time required.....	2 hr., 15 min.	2 hr., 20 min.
Length of weld.....	55 in.	48 in.

Welding Tension Test Specimens

Electrical (of arc) as measured at machine.....	195 amp., 20 volts	165 amp., 20 volts
Time required.....	2 hr., 15 min.	2 hr., 15 min.
Length of weld.....	55 in.	44 in.

Results

Bend Test

Vertical member of T-bar laid flat against flange; assembly showing no signs of failure

Tension Test

Specimen number	Maximum load, pounds	Distortion of T after failure, inches	Remarks
1	12,000	$\frac{1}{32}$	Bead tore away from armor
2	13,000	$\frac{1}{16}$	Broke in weld on one side; bead pulled off armor on other side
3	14,750	$\frac{3}{16}$	Broke through weld
4	10,750	$\frac{1}{8}$	Broke through weld on one side. Failure on other side partly due to bead leaving armor and partly to weld leaving bead
5	10,250	$\frac{1}{8}$	Bead tore away from armor
Average.....	12,150		

TABLE 5.—TEST NO. 5, OBSERVATIONS AND RESULTS

Desired Conditions

	Welding bead to armor	Welding T-bars to bead
Arc amperes.....	145-150	135-145
Speed of deposition.....	2 ft. per hour	2 ft. per hour
Method of welding.....	Deposit one electrode and skip 1 ft. before making next deposit	

Observations

Welding Bend Test Specimens

Electrical (of arc) as measured at machine.....	145 amp. 18 volts	140 amp. 18 volts
Time required.....	2 hr.	2 hr., 5 min.
Length of weld.....	49½ in.	48 in.

Welding Tension Test Specimens

Electrical (of arc) as measured at machine.....	145 amp., 18 volts	140 amp., 18 volts
Time required.....	2 hr.	2 hr., 5 min.
Length of weld.....	51 in.	44 in.

Results

Bend Test

Vertical member of T-bar laid flat against flange; assembly showing no signs of failure.

Tension Test

Specimen number	Maximum load, pounds	Distortion of T after failure, inches	Remarks
1	10,500	$\frac{1}{16}$	Broke through weld on one side
2	12,250	$\frac{1}{16}$	Bead pulled away from armor
3	12,000	$\frac{1}{8}$	Bead pulled away from armor
4	12,250	$\frac{1}{4}$	Broke through weld on one side
5	11,000	$\frac{1}{4}$	Broke through weld on one side
Average.....	11,600		

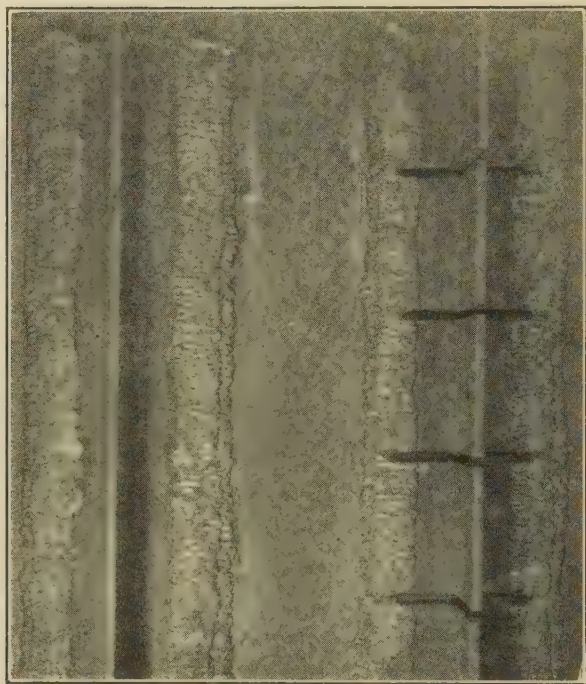


FIG. 8.—Test No. 1, after completion of welding and before tests to destruction.
Bend specimen at left, tension specimens at right.

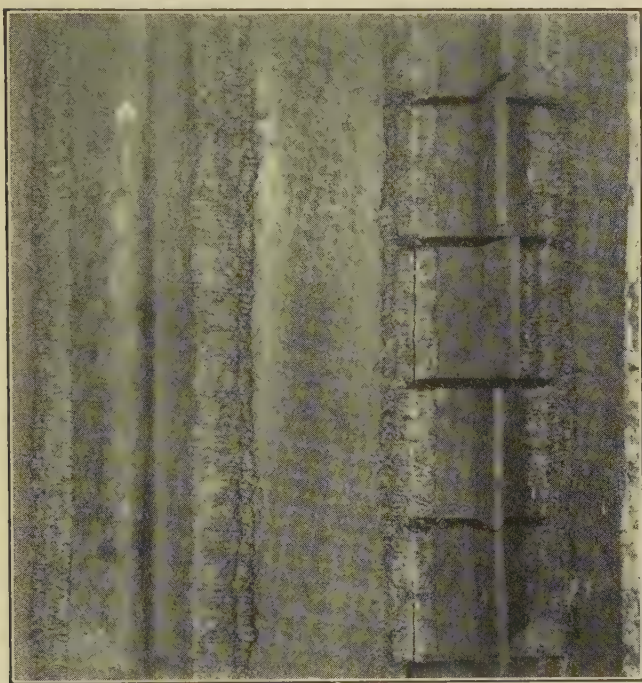


FIG. 9.—Test No. 1, after completion of bend and tension tests.
Bend specimen at left, tension specimens at right.

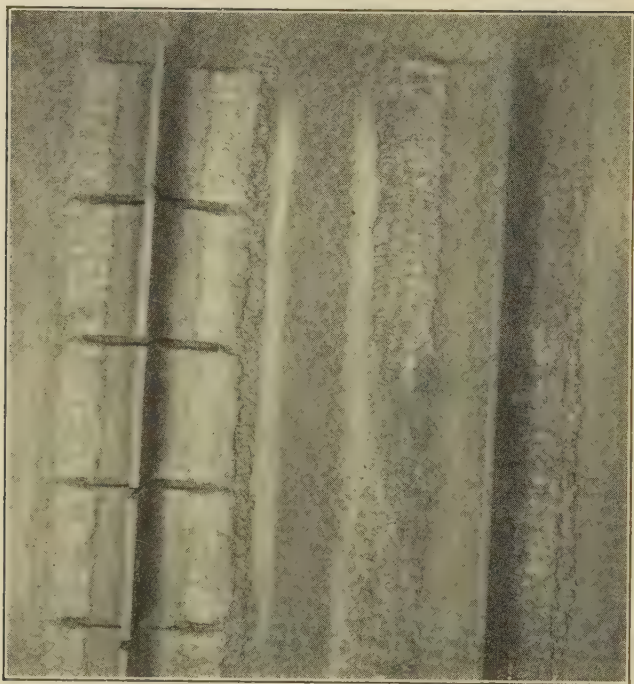


FIG. 10.—Test No. 2, after completion of welding and before test to destruction.
Tension specimens at left, bend specimen at right.

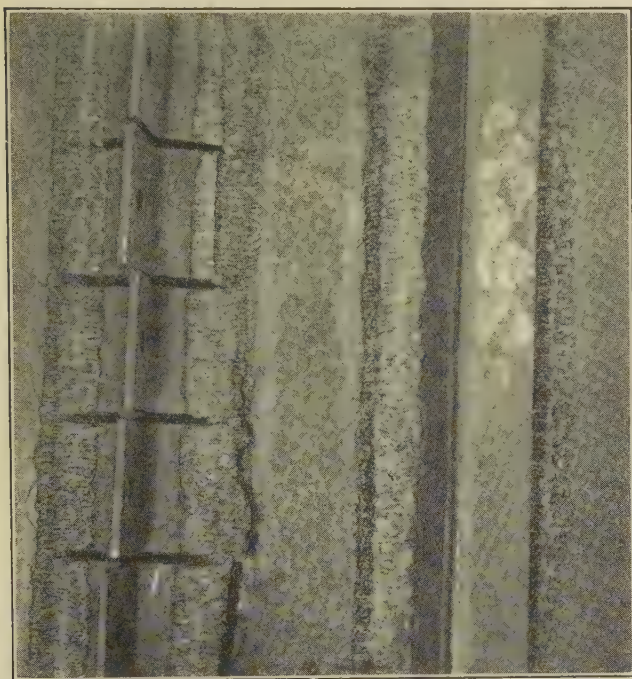


FIG. 11.—Test No. 2, after completion of bend and tension tests.
Tension specimens at left, bend specimen at right.

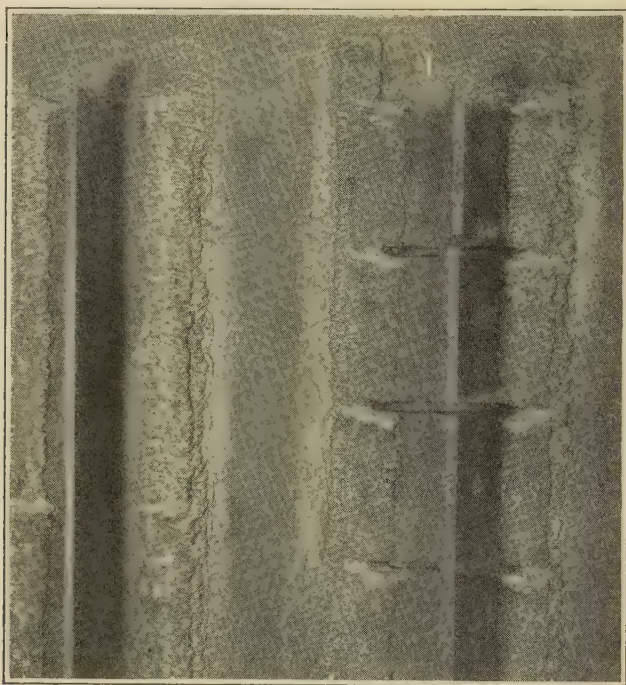


FIG. 12.—Test No. 3, after completion of welding and before test to destruction.
Bend specimen at left, tension specimens at right.

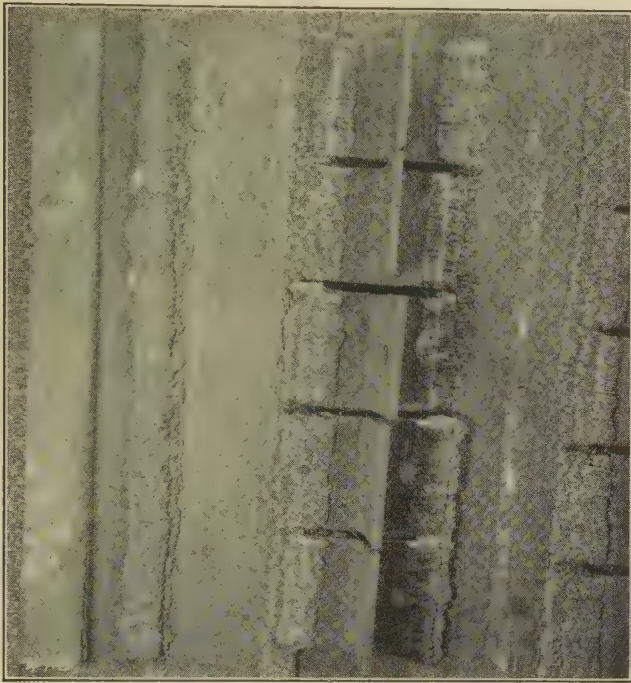


FIG. 13.—Test No. 3, after completion of bend and tension tests.
Bend specimen at left, tension specimens at right.

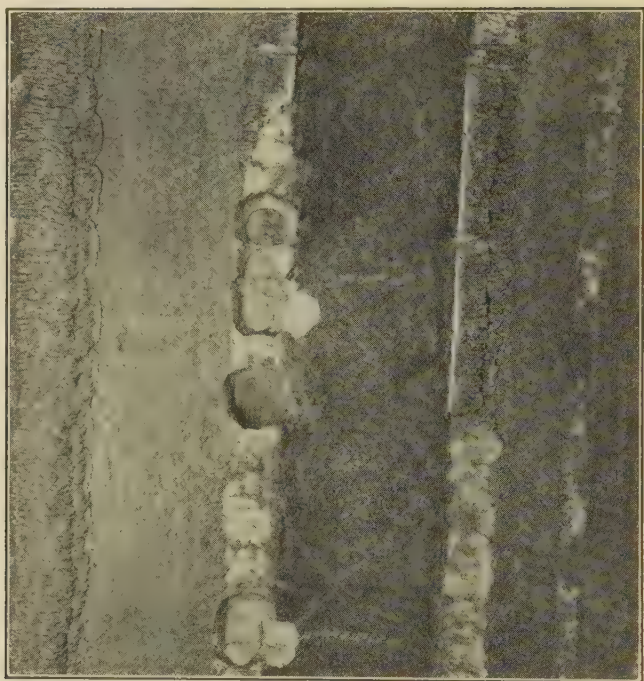


FIG. 14.—Test No. 3, spalling which occurred in the armor plate during the arc-welding operations.

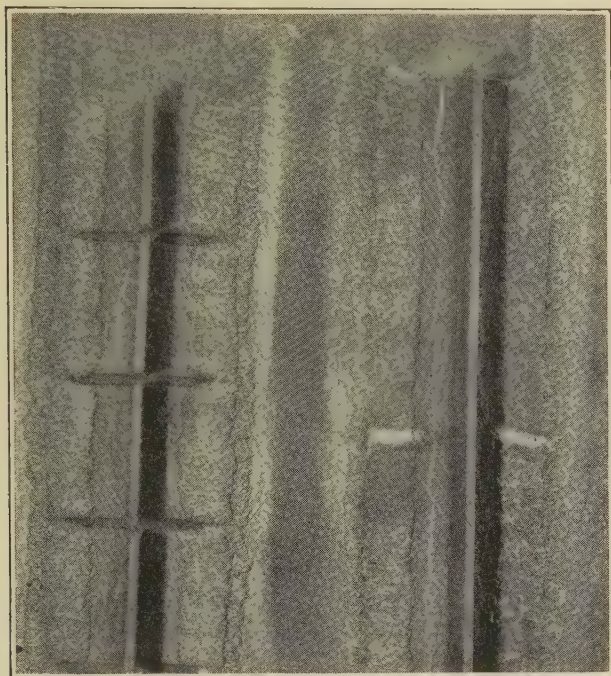


FIG. 15.—Test No. 4, after completion of welding and before test to destruction.
Tension specimens at left, bend specimen at right.

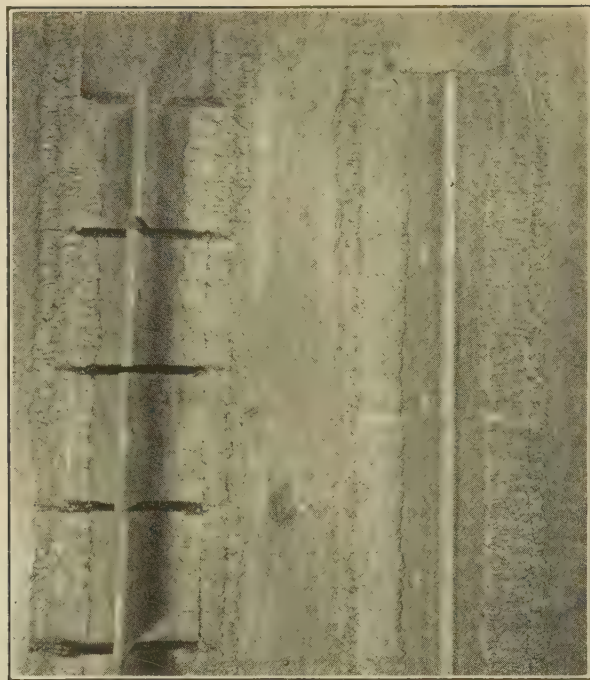


FIG. 16.—Test No. 4, after completion of bend and tension tests.
Tension specimens at left, bend specimen at right. The vertical member of the T-bar broke during bend test.

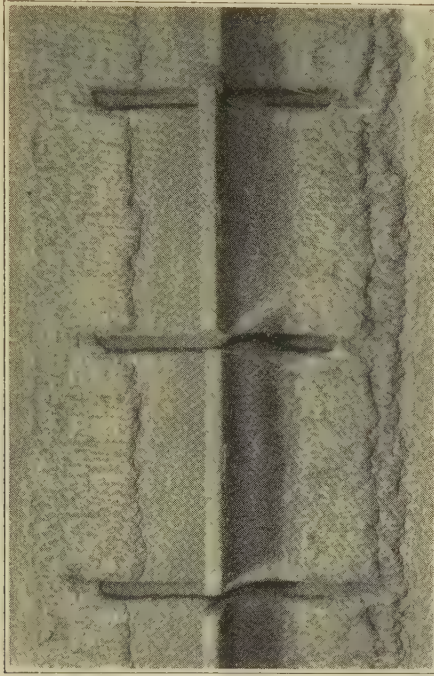


FIG. 17.—Test No. 5, after completion of welding and before test to destruction.
Tension specimens shown.



FIG. 18.—Test No. 5, after completion of bend and tension tests.
Tension specimens at left, bend specimen at right.

CHAPTER III

DISCUSSION

In all tests after depositing the bead to the armor, there was evidence of slight cracks, but after welding the T-bar to the bead, these cracks became more apparent. The cracks seemed to occur to an equal extent in all tests except test No. 3, where they seemed to be more numerous and pronounced. This was confirmed by the fact that only in test No. 3 was it possible to knock out the pieces of armor surrounded by cracks.

Considering failure by tension, it would seem that the method used in depositing the bead on the armor for test No. 1 gave the best results; that is, all tension-test specimens broke in the weld rather than the bead tearing away from the armor. The average strength at failure for this test was 11,000 lb.

The deformation of nearly all tension-test specimens, as shown on Fig. 19, indicates that up to a certain load the weld is stronger than the T-bar, but that after a certain amount of permanent deformation has taken place, the redistribution of the load is such that the weld becomes weaker than the T-bar. ✓

Practical Application.—After considering the deformation which had taken place in nearly all the tension-test specimens, additional tension-test specimens were prepared, by machining the T-bar from $4\frac{1}{2}$ in. across the flange to $2\frac{1}{2}$ in., in order that the application of the weld would approximate the location of the rivets in riveted construction. These specimens were welded to the armor plate and tested in tension. The results, as given in Table 6, test No. 6, indicate that the arc welding is stronger than riveted construction, providing it is properly applied (Figs. 20 and 21). ✓

The blisters on the U.S.S. "Arkansas" and U.S.S. "Wyoming" were attached by using the methods given in test No. 1, and the completed structures when submitted to the required hydrostatic and air tests developed no weakness from non-water-tightness or strength. ✓



FIG. 19.—Deformation of many of the T-bars under tension.

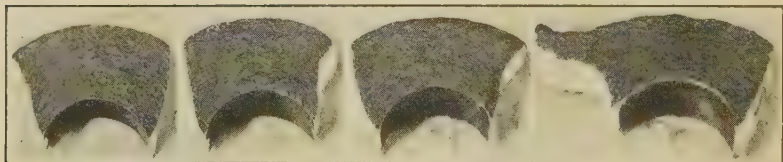


FIG. 20.—Test No. 6, showing pieces pulled out of modified T-bars during tension tests.



FIG. 21.—Test No. 6, showing modified T-bars after completion of tension test.

Conclusions.—Various methods for attaching the blisters were suggested, but after the foregoing test was completed they were discarded, because of the low cost, ease of application, and satisfactory structural values obtained by arc welding.

TABLE 6.—TEST NO. 6, RESULTS
Welding Conditions

	Welding bead to armor	Welding T-bars to bead
Arc amperes.....	165-170	150-155
Speed of deposition.....	2 ft. per hour	2 ft. per hour
Method of welding.....	Deposit one electrode and skip 1 ft. before making the next deposit.	

Results of Tension Test

Specimen number	Maximum load, pounds	Remarks
1	27,700	Failure occurred by pin shearing through web
2	32,300	Failure occurred by pin shearing through web
3	30,500	Failure occurred by bead tearing away from armor on one side; web of T-bar badly deformed at failure
4	23,500	Failure occurred by pin shearing through web
5	19,300	Failure occurred by pin shearing through web

In preparing the foregoing, the authors desire to express their appreciation to the welders at the Philadelphia Navy Yard for their suggestions and cooperation in this work, to the drafting rooms where the plans were prepared, and to F. J. Cleary, Commander U. S. N., for his assistance and suggestions.

PART VII

ARC WELDING OF DUPLICATE STEEL STRUCTURES

BY

W. H. HIMES

CHAPTER I

PRELIMINARY PLANNING

The application of welding methods to the construction of truck switches required rather more daring than might generally be supposed. To appreciate fully the problem involved, an understanding should be had of the drastic interchangeability requirements in the operation of this type of power-house equipment. Fundamentally, the truck type of switch gear is safety apparatus. The practice for many years has been to mount circuit breakers permanently behind the switch panels, attaching them permanently to the busses and leads through disconnect switches. Thus any repairs to the breaker involved an element of danger to the operator, as well as loss of the use of a certain section of the busses during the repair.

In the truck type of switch gear the breaker and auxiliary apparatus are mounted on a portable frame. The wheels on which it rolls are suitably flanged to engage guide rails leading into a cell or compartment abutting against the bus line. Bus and lead contacts are so disposed at the rear of each cell that any one of a group of the portable units may be rolled into any cell and proper contact automatically established between the bus line and the particular lead through this portable truck unit. By this system any breaker may be readily removed for inspection or repairs. The repairs may be made in perfect safety, and during any repairs the particular lead in question may be serviced by means of a spare truck unit, with only a momentary interruption while interchanging trucks. Frequently there are 30 or more of these interchangeable units in one structure. When an emergency arises in a large power station at, say, two o'clock in the morning, with but one operator on the floor, the interchangeability must be a very real condition.

Furthermore, the cell structures, which are factory made and shipped ready to bolt to the power-house floor, must not only adjoin each other exactly so as to present a neat front when the trucks are in place, but they also must align with the holes

in the power-house floor or overhead structure previously prepared for the individual leads for each breaker. Thus these structures have to be made very exactly to dimension, for the installation must be put in the minimum space.

Thus the requirements are maximum of mechanism in a given space, assured alignment with previously installed lead openings

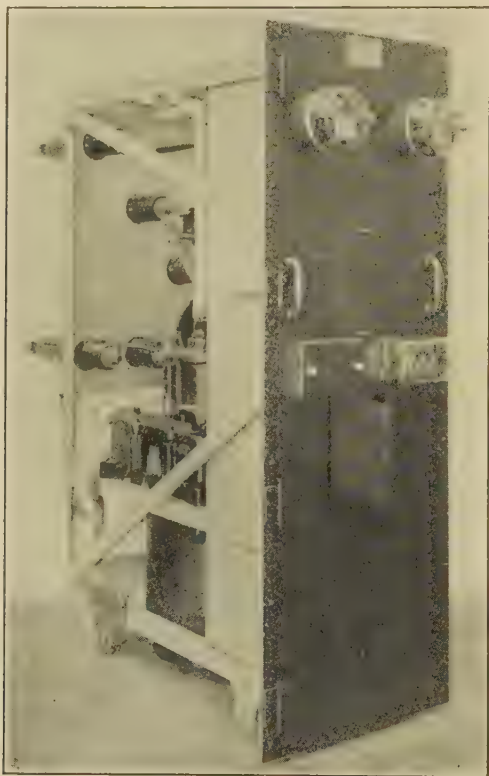


FIG. 1.—Front view of truck switch.

and foundation holes, uniform front appearance, and assured interchangeability of all trucks in all cells. This little matter of interchangeability involves the proper engagement between the portable and stationary units of at least 6 main contacts, from 12 to 50 auxiliary contacts, a ground contact, an automatic shutter mechanism, and a leverage mechanism for forcing the truck into position against the resistance of the contacts. Thus it appears that the cells and also the truck structures must be

closely mated duplicates of each other. Typical trucks and cells are illustrated in Figs. 1 to 7.

When this line of switch gear was first introduced, welding was tried tentatively and abandoned, as the shop reported that no two structures could be made alike on account of warpage. Riveting was also tried and abandoned. The structure that

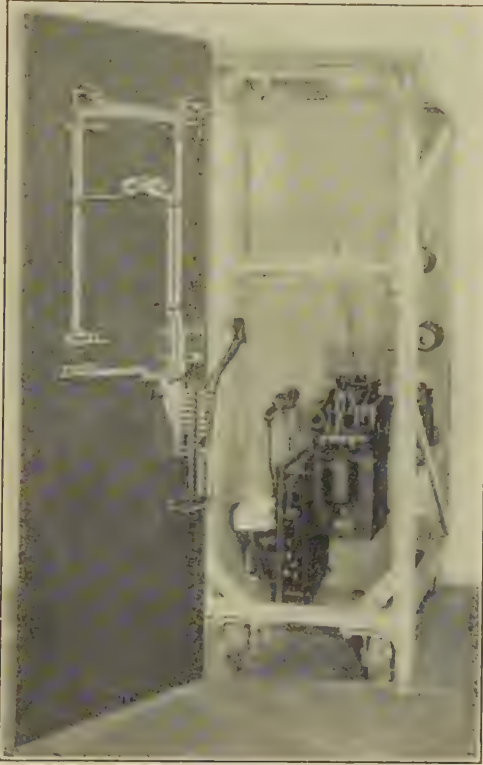


FIG. 2.—Interior of truck switch.

finally was developed, and which could be produced interchangeably, was a rather complex bolted frame of angle iron and malleable corner castings having machined surfaces. This construction although fairly effective was expensive, and when a redesign was determined on, a careful study of requirements, as well as various types of construction, was made. In spite of the conviction in the shop that welding was out of the question, the admitted advantages of welding were many, and a tentative

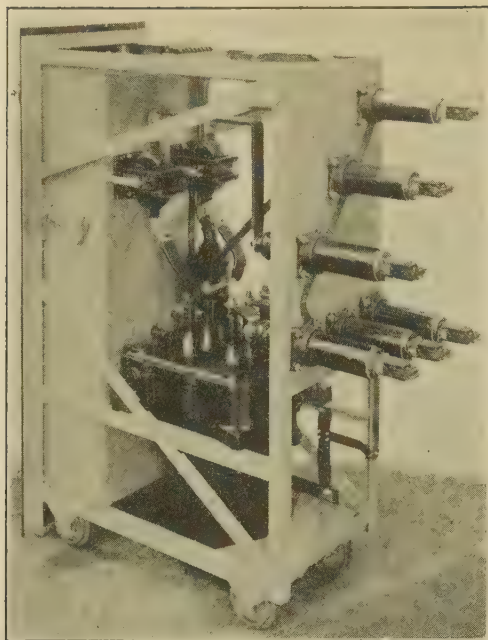


FIG. 3.—Rear view showing main contacts.
The out-turned front flange is seen at the extreme left.

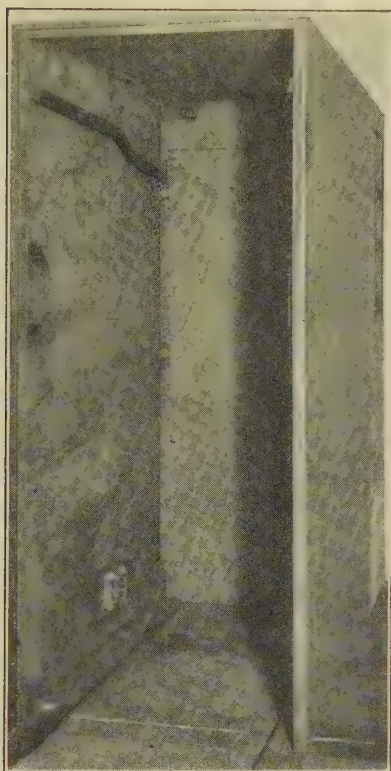


FIG. 4.—The first welded cell.
Welded cam is shown on wall at left side.

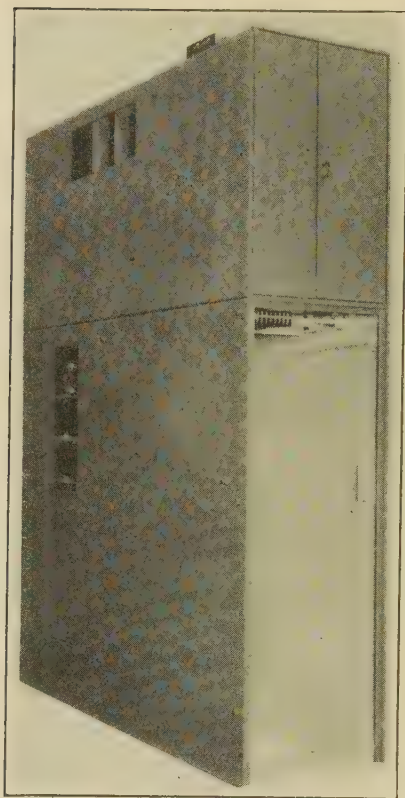


FIG. 5.—Completed truck structure.
The superstructure is not discussed.

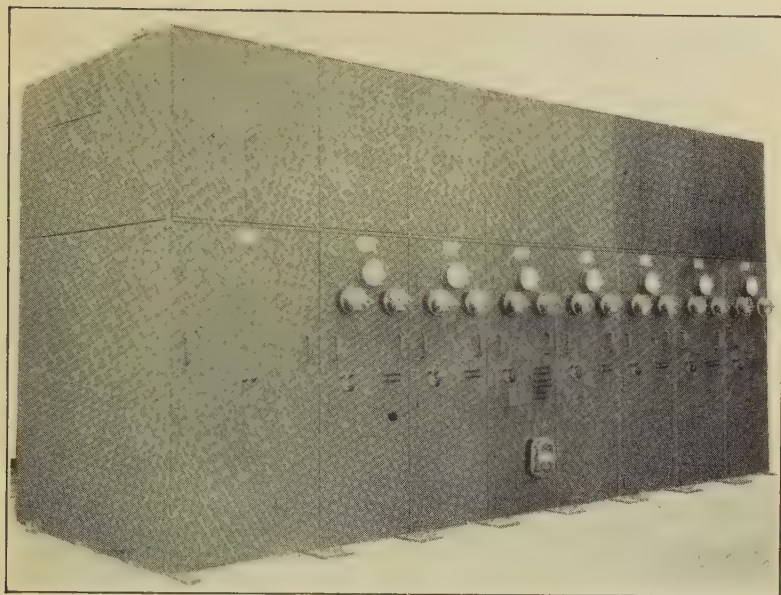


FIG. 6.—Complete switch set with trucks in place.

design of welded structures for both truck and cell was worked out. The chief requirements involved were:



FIG. 7.—Results of impact test.

1. Economical manufacture.
2. Assured interchangeability.
3. Strength for rigidity in shipping and service.
4. Ease of erection and service.

THE FIRST EXPERIMENT

[illegible]

the elements while they were clamped to a heavy bedplate. The stresses to which the frame was to be subjected rendered it desirable to provide bracing similar to that shown in Fig. 8. The complete truck structure is illustrated in Figs. 1 to 3. The various elements shown in this figure were clamped securely to

the bedplate, and the welder started around the structure, finishing the welds in rotation. When finished, it was found that while the clamping had provided a frame that would lie fairly flat on the table, the front member *a* was drawn out of true in the plane of the figure by the diagonal braces. Repeated trials developed the fact that the nature of the deflection could be altered by the sequence of the welds. The requirement for a satisfactory panel were simply that it should lie flat on the bedplate when free and that the front member *a* should be so straight that its out-turned flange would fit snugly against the cell edges when the complete truck was pressed into position. This meant a maximum variation from normal of $\frac{1}{16}$ in. The rear uprights might depart from the vertical by $\frac{1}{8}$ in. in their length without serious results. Also, an exact right-angle for the horizontal members was not necessary. Flexibility in this regard had been provided by the new design of adjustable wheel brackets.

Thus the welding problem consisted in finding the sequence that would throw the least distortion into the front member. This sequence was worked out and was found to give excellent results.

CHAPTER III

THE BEST SEQUENCE

The lower diagonal was first welded to the horizontal member at *b*, Fig. 8. While this was cooling, the upper rear corner was welded at *c*, and next the lower rear corner at *d*. Returning to the upper corner, the diagonal was welded at *e*. The remaining joints were welded in the sequence indicated by the letters. It will be seen that by the time *h* is finished, the rear upright has been completely tied into the cross-members in such a sequence as to reduce to a minimum the concentration of welding heat. Also, the free ends of the cross-members were free to creep under the clamps to adjust themselves as to length as the welds cooled. Before weld *g* was made, which was the first tie into the front member, the weld *c* at the opposite end of the cross-member had been cooling during the welding of *d*, *e*, and *f*. Also, during the welding of *b*, *f*, and *h*, which closed a rigid structural triangle, the ends *k* and *l* were clamped, so as to hold them tightly against the bed, but permitting free creepage in all directions during the heating and cooling cycle at the other end. These intersecting braces had been one of the chief causes of trouble in the first trials. However, by the new sequence, welds at *i*, *j*, and *k* gave *h* time to cool before *l* at the other end of the diagonal was started. In this way frames that would lie flat on the bedplate were produced, which had front members 65 in. long that came within $\frac{1}{16}$ to $\frac{1}{32}$ in. of being straight.

The next and obvious step was to combine a right-hand and a left-hand frame so constructed into a cubical structure by welded connecting members. While being so welded the pair of frames were held rigidly in position by very strong braces. The same general principles were followed as with the frames, and especially the avoidance of excessive heating owing to too prolonged welding work in one place.

CHAPTER IV

STRUCTURAL DESIGN

Especial attention was given to the design of the different faces of the frame to insure that diagonal bracing should be included in each face of the frame.

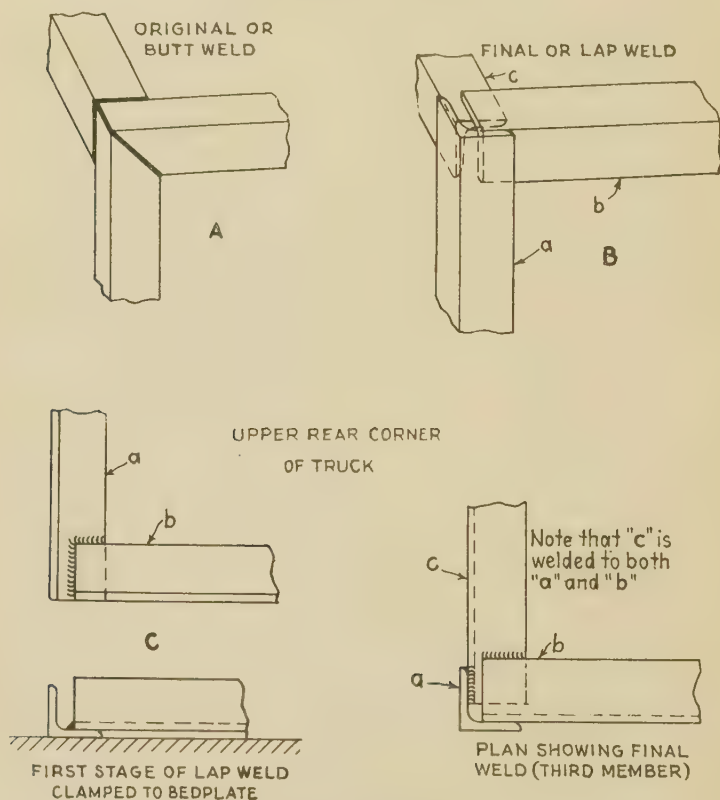


FIG. 9.—Details of joints.

Theoretically a cubical structure can be shown to be rigid if five of the six faces are braced against distortion in their own planes. However, since accessibility required that several of

the faces should have generous openings, full bracing as understood in structural design was not possible. In front, for instance (Fig. 2), only short diagonal bracing at the four corners could be permitted on account of the meters and other apparatus projecting through openings in the switchboard panel. Therefore as much bracing was inserted in each of the other five faces as possible, and altogether a very rigid structure was produced.

When the first sample was built, it was felt that the original construction should be followed in general so as to disrupt the electrical design as little as possible; thus the angle members were kept flush at the joints, as in the previous practice. To do this a joint as shown in Fig. 9, detail A, was tried. The miter joint was used on the side panel and the connecting pieces were butted with about $\frac{1}{8}$ in. weld space ($\frac{1}{4}$ in. angle thickness).

CHAPTER V

FINAL TYPE OF JOINT

It was realized that butt joints were harder to make, and, after an order was constructed in this way, a study of the apparatus installed showed that there was room enough to permit lapping at the eight corners.

This change produced beneficial results. The lapped members could be sheared to a $\frac{1}{16}$ -in. tolerance, sometimes $+0 - \frac{1}{8}$ in. Fillet welds were possible instead of butts, which reduced the distortion owing to shrinkage of the weld metal. In fact, the distortion of the front angles was practically eliminated when this change was made. Considerable thought was given to providing joints of such nature that overhead welding was reduced to a minimum. Figure 9, *B* and *D*, shows a typical corner weld of the lap type. Detail *B* is a perspective view of the complete weld, showing all three members tied at the top rear corner of the truck. Detail *C* shows members *a* and *b* lapped on the bedplate for the first stage of the assembly in forming a side panel, and also shows flexibility as to length of pieces. Detail *D* shows a plan view of the same corner raised to an upright position with angle *a* vertical and the cross-member *c* added to connect the two panels. Notice that the weld metal is deposited downward in both stages. Similar procedure was worked out at the other corners.

An important development in this method of assembly is that several capacities of apparatus may be housed in the same pair of side panels by connecting them into different widths of cubicle. Once standardized, these panels may be made up in quantities and stocked for future assembly according to the orders received, whereas stocking of the assembled cubicle is impractical.

The diagonal bracing on the sides was a more difficult problem. There was so little clearance for the apparatus that these members could not be lapped, and the butt weld was continued. The short diagonal braces in the front opening were also put on by means of butt welds, there being no other way apparent at the time (Fig. 2).

CHAPTER VI

FIXTURES

Mention has been made of the bracing of the first cubical structure during welding. This was by means of temporary

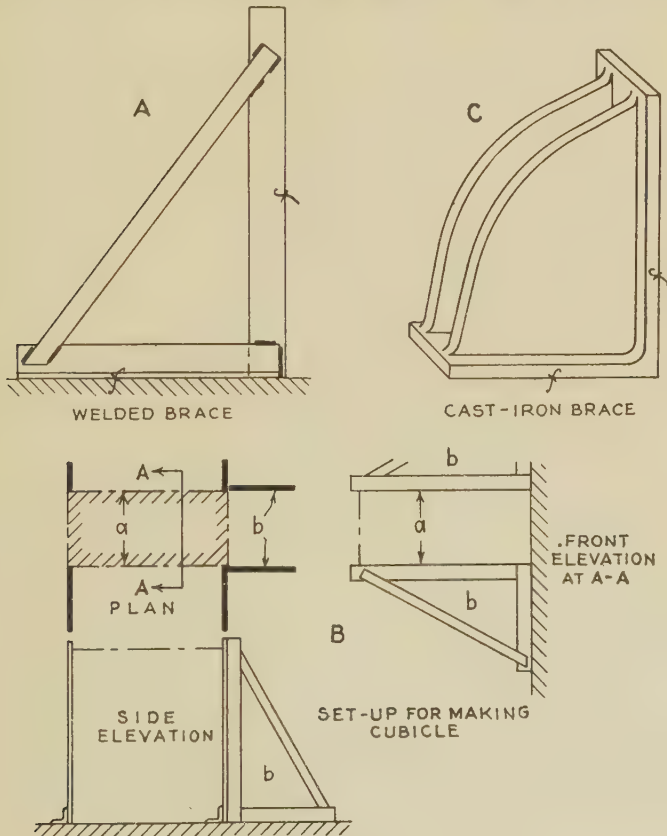


FIG. 10.—Types of bracing.

braces, but when production was started a series of triangular frames was made up as shown in detail A, Fig. 10. These could be locked rigidly to the bedplate. Two pairs of these were set

up facing each other on a bedplate, as shown in detail *B*, with the dimension *a* certified as being the correct width of the truck cube. Two other braces *b* were located at the front. The front edges of the right and left panels were clamped against these latter braces, insuring parallelism of their front edges. The panels are also clamped or wedged against the front and rear side braces, and the connecting pieces are then welded in place. When the frame is finished, it is drawn vertically out of the fixture by the crane. Thus the fixture is not disturbed nor readjusted after being set up.

Figures 1 to 4 show the details of the various joints of the approved structure.

✓ An incidental exhibit of the economy of welding as applied to shop fixtures is seen in a comparison of two sets of fixtures. The six braces referred to were fabricated by welding structural shapes. A set previously made for a smaller sized unit was made of cast iron. These are shown in Fig. 10, detail *C*. The cost of patterns, castings, and machining of this set of eight amounted to over \$800, while the welded set was built for less than \$150. The latter set was more accurate, of much greater capacity, and gives the welder better access to the work. ✓ The welded fixtures were constructed in less than one week, while the cast-iron set was two months in process.

As to the clamping mechanism used for holding the angles to the bedplate while they are welded, T-slots are very expensive to machine and bolt clamps are slow to operate. When a new frame size is to be welded, the positions of the required clamps are marked and plain holes are drilled in the bed at these locations. A clamp called a "figure 7" is used. These are of round stock, having fair clearance in the holes ($\frac{1}{64}$ in. in a $\frac{3}{4}$ -in. hole) and shaped as shown in Fig. 11. When dropped in place with the horizontal leg on the part to be clamped, one or two blows of the hammer will suffice to lock them securely in place. To disengage, a blow of the hammer against the back of the clamp will immediately release it. The device is not new, but was borrowed from the shipyards, where it is used to hold ship frames in place while being bent. They are at least four times as fast as any screw clamp and very cheap.

The theory on which the clamp grips is generally familiar in other applications. There are certain limitations in its design for successful operation which might well be pointed out, otherwise a shop man might go to some expense in making equip-

ment that would not work. Of course the limiting quantity to be watched is the coefficient of friction between the clamp and the hole in the bedplate. The reaction of the plate to be held may be indicated by the vector W (Fig. 11, *B*). This reaction causes a turning moment Wl . The moment is resisted by the edges of the hole at m and n . These resistances have a leverage of L . Hence, as these resistances constitute the balancing couple preventing rotation, LR must equal Wl , and R and W are inversely proportional to these respective leverages. R will be very large as compared to W , if the design is practical. If ϕ is the angle of friction, then f is proportional

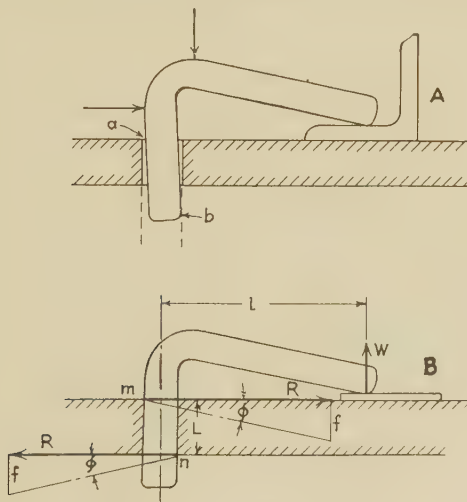


FIG. 11.—Use of "figure 7" brace.

to the frictional resistance at one edge. Whenever $2f$ exceeds W , we have a locking condition. If, however, the plate is so thick or the arm of the clamp so short as to reduce R so that its resultant $2f$ is less than W , then W is unbalanced and the clamp will rise in the hole after each hammer blow. An average value for f seems to be 15 per cent. When the bedplate is so thick that the stem of the clamp does not reach the bottom of the hole, care should be taken to have the end of the stem tapered at least on the contact side, so that it will not gouge into the surface of the hole at b when struck by the hammer (Fig. 11, *A*). For the same reason, the hole should closely approximate the size of the stem or there will be excessive resistance to driving at a .

CHAPTER VII

THE CELL

The specific discussion preceding has applied to the portable or truck structure. The stationary receptacle is also supplied by the manufacturers. As previously designed, this was also of the bolted angle and corner-casting design.

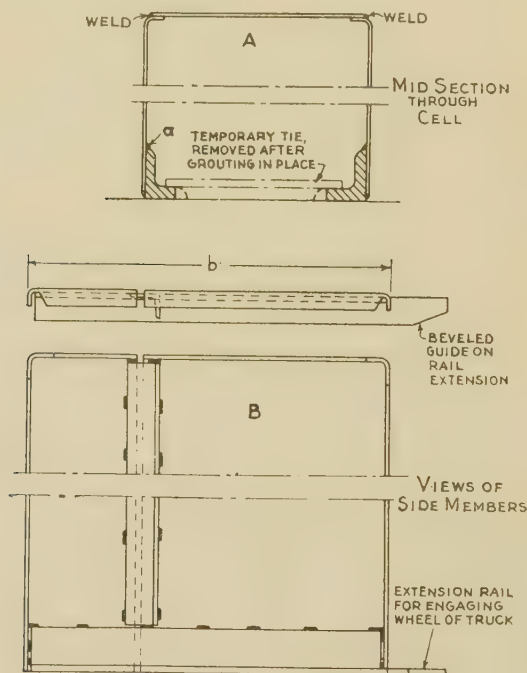


FIG. 12.—Details of joints.

Two types of welded cell were considered. First, an angle-iron frame covered with sheet iron, and second, a welded sheet-steel cabinet depending as far as possible for rigidity on the flanges bent into the sheets at the corners. A compromise was developed in which the sturdy angle-iron rails of the old design were retained. These were made of 4- by 5- by $\frac{5}{8}$ -in. angles and

performed the double office of a base on which the upper structure was built and of guide rails for the wheels of the portable unit. Figures 1 to 4 show the general construction of the cell, while the details of the joint, etc., are further shown by Fig. 12. Some apprehension was felt regarding the welds at joint *a*, detail A. As this was between $\frac{1}{8}$ - and $\frac{5}{8}$ -in. stock there seemed a possibility that the thinner metal would melt through before the heavy stock had fused. The welders experienced no difficulty.

As with the truck, the two sides were assembled first. A side is shown in Fig. 12, *B*. The object of the vertical angle is to support a removable diaphragm or partition separating the live bus circuit from the breaker compartment. As the distance *b* exceeds the standard width of such sheets, this angle makes a convenient position for a joint. With the two sides complete, the procedure is similar to the truck practice. Rigid braces are used to hold the sides in position while the connecting parts are welded in place. It is not necessary to weld continuously along these joints, so a system of "skip" welding was worked out. Welds of 1 to $1\frac{1}{4}$ in. were made every 6 in. or so. These give ample strength and do not overheat the metal. A 1-in. weld is about the shortest that can be relied upon.

In general, much less difficulty was encountered with the cell than with the truck. The shop soon learned to make the opposing flanges very closely to dimension. Also, the laps on the vertical angle and on the horizontal base angle gave a chance to "go and come" to secure the correct overall dimensions.

Several details of the cell are of interest in connection with welding. There is the shutter mechanism which protects the live contacts while the truck is out of the cell. One of the

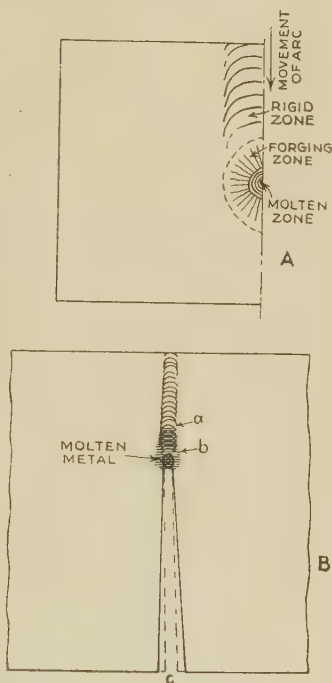


FIG. 13.—Sketch showing origin of strains.

shutter arms is visible in Fig. 4. Its general shape is as shown in Fig. 14, *A*. This arm is pivoted at *a* to the front of the cell, while *b* is linked to the shutter, which opens by rising. The shape of the arm was so developed that a roller on the back corner of the truck—see truck photos—will engage the bottom surface of the arm and begin to raise the shutter at about mid position, as at *c*. In order to provide for the free entrance of

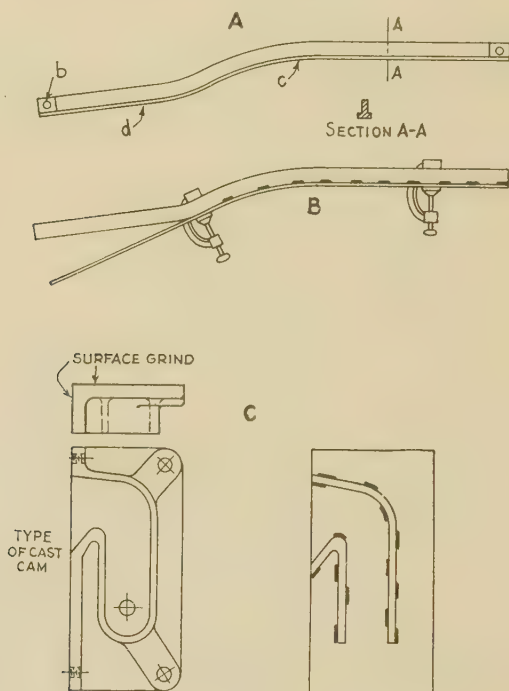


FIG. 14.—Detail of shutter arm.

the contacts, the shutter must be fully opened at *d* and the remainder of the roller travel must produce no further motion of the shutter.

In previous designs similar arms had been made of malleable iron. In the development of the new design there was not time to make patterns, so the arm was fabricated in a few hours by welding. The exact contour was cut from a sheet by the cutting torch as shown in Fig. 14, *B*. But a $\frac{1}{8}$ -in. strip 4 ft. long would have no rigidity. Some style of horizontal flange was necessary. As the roller must engage the bottom edge, an inverted T-section

seemed ideal. So a strip of $\frac{1}{8}$ by $\frac{3}{4}$ in. stock was clamped and tack welded as shown in *B*. As the welding progressed, the clamps were moved along until the flange was completely tied to the web. In this way several different contours were tried out with very little expense.

When the first order was taken, it was for too short a time to make malleable castings. But on all previous designs the cam that engages the lever arm for forcing the truck home against the contacts had been made of malleable iron. This first order was filled by welding the cams, as shown in sketch 7, *C*. When the order was finished, a cost analysis showed that the welded cam could be fabricated at a considerable saving over the malleable casting.

In all previous structures the variation in width of individual cells was absorbed by a special threaded sleeve and tie bolt. The previous allowance for variation had been $\frac{1}{4}$ in. After a few orders of welded cells had been installed it was found that even the $\frac{1}{8}$ -in. allowance provided in the welded cells was not necessary. The complicated adjusting sleeve was eliminated, and a plain $\frac{1}{8}$ -in. washer was substituted between adjoining cells.

CHAPTER VIII

RESULTING ECONOMIES

It is of interest to note that the original bolted truck structures were of $2\frac{1}{2}$ - by $2\frac{1}{2}$ - by $\frac{3}{8}$ -in. angle. The first welded truck sample was made of $2\frac{1}{2}$ - by $2\frac{1}{2}$ - by $\frac{1}{4}$ -in. angle, and with the bracing employed it was so rigid that all subsequent jobs were made of 2- by 2- by $\frac{1}{4}$ -in. angle, thus making a considerable saving on the steel alone. By the old process careful layouts were made by skilled men, marking by different symbols the holes to be drilled, tapped, or countersunk. This marking took time and in spite of care was frequently misinterpreted by the driller, with scrappage resulting. By the new process, the angles are sheared to $\pm \frac{1}{16}$ in., and the only drilling required is for securing the various pieces of apparatus to the truck. The welder clamps the pieces against certified stops in his fixture. The overlap of the angles absorbs the variation in the shearing, and the fixture assures absolute interchangeability. The resulting frames are ready for the next assembly stage (putting on wheel brackets) without any bending, springing, or subsidiary process, as was the case previously.

Incidentally, inspection expense at this point was considerable. The frames were rarely perfect as first bolted, and careful inspection at this stage was necessary. At present, however, no inspection at this stage is required other than approval of the first unit of an order.

Of course all of the glory for this cost reduction cannot be attributed to the welding process. When the redesign was undertaken, it was found that many drastic manufacturing tolerances had been gradually built up in order to secure the interchangeability. For instance, all these truck structures have a height from two to three times the wheel gage; thus any error in height of wheel bracket or wheel diameter was multiplied by two or three in the extent to which the structure would be out of plumb. The limit held by the inspectors was $\frac{1}{32}$ in. in about 6 ft. Therefore, the diameter of wheel, height of bracket,

and trueness of bracket seat had to be held to very close tolerances, and, at that, much adjustment was required after assembly in order to meet inspection requirements.

So when a welded design was considered, "automobile assembly methods" were abandoned as being unsuited to the requirements. It was fully expected that the welded cubicle would be more difficult to hold to shape and size than the bolted one, and so the wheel bracket was provided with universal adjustments. After the wheels and brackets are assembled to the welded frame, the wheels can be centralized to the frame (as to tread) and the gage between wheel flanges also adjusted to suitable templates. Also, the wheel bearing is eccentric to the pin fit in the bracket, so that by rotating the individual wheel pins any condition of height may be obtained to plumb the truck frame. In this way the tolerances on the wheels and brackets were changed from $\pm .001$ in all height dimensions to $\pm \frac{1}{64}$. Anyone familiar with production costs will appreciate the saving involved here.

This is a good illustration of a very important point in the introduction of welding methods. It is not practical merely to substitute welding for bolts and nuts without real engineering supervision and the adaptation of the design to the new process; therefore, it is equally obvious that a thorough understanding of the limitations of the welding process is essential to permit its successful use.

In the present instance, however, it had been found that the "tolerance" manufacturing method was unsuitable even with bolted construction; therefore the abandonment of this method was nothing against welding. In fact, the structures as they now emerge from the fixture are much better suited to exact assembly methods than were the previous structures.

In the truck and cell together, \$12 worth of hardware (nuts, bolts, etc). was eliminated by the change. Also, 20 malleable corner castings were being surface-machined on three sides, as well as drilled, and in some cases tapped. A saving on both structures that is impossible to state accurately is in the reduced inspection and adjustment expense, which was quite a factor in the old design.

An advantage that cannot be evaluated in dollars is the reduction of time in process. Layout and drilling time being

eliminated, assembly starts as soon as the sheared material is put in the hands of the welder.

A glance at any of the photos (Figs. 1 to 4) will show that the supporting structure involves a small part of the cost of the complete unit with all of the electrical apparatus installed. However, the economies in the welded structure were so marked as to remove the activity from a position very close to the line between profit and loss to a condition where some profit is shown, and this without sacrifice, but with improvement in the quality of the product.

CHAPTER IX

CONCLUSION AND APPENDICES

In the foregoing, the problems encountered in a specific application of arc welding to a manufactured product have been discussed. The means and methods of solving the problems involved have been set forth. Much additional detail and theory have been covered in the four appendices. It is believed that the successful application of arc welding in this apparently difficult case will be encouraging to those manufacturing similar material.

APPENDIX A

A discussion of the elementary mechanics of rigid structures is hardly in order, and yet there are a number of draftsmen who attempt to build such structures without diagonals and who attempt to stiffen their wabby structures by adding more parallel members.

In a rectangular frame, as shown in Fig. 15, outside forces tending to distort the frame may be represented by the vectors of magnitude E at m and n . The frame as shown has only the resisting power inherent in the metal at the corners. The metal at o and p will receive the maximum stress. If for the sake of simplicity we assume a total concentration of stress at these points, then the necessary resisting power at these points to maintain rigidity would be $\frac{\bar{F} \times a}{b}$. The applied force would cause a stress inversely proportional to the two lever arms. As shown in the sketch (Fig. 15) the forces produced would be about eight times as great as those that cause them. However, as they may be assumed to be equally distributed at the four corners, we have the ratio of 8:4 to contend with. Assume now that an additional cross-member were applied, we would have the resisting work distributed over six joints instead of four; the two new ones would only have a leverage of k instead of b , so that the resulting strains would not be reduced as much as might be expected.

If the diagonal is introduced as shown at *c*, the external forces will clearly be absorbed by this diagonal directly, relieving the four corners of all strain. Only enough strength now is required at the corners to insure cohesion of the two members adjoining. If the diagonal be introduced into the frame as shown at *d*, practically all stress is still removed from the corners. The bond between the corners and the diagonal must now be strong enough to transmit a force exactly equal to the applied external forces, but in tension.

From the foregoing it should be clear to one not a structural designer that diagonal bracing is much the more efficient. Of

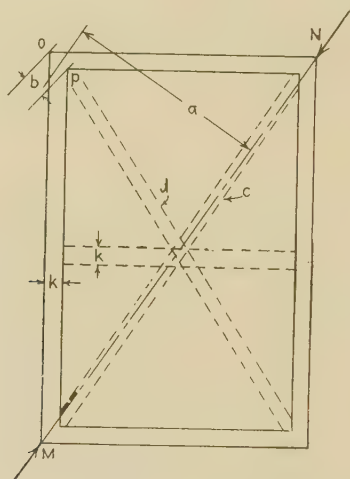


FIG. 15.—Forces producing distortion.

course, when a sheet may be used filling in a panel or side, this sheet is highly efficient, as it includes the diagonal metal in itself.

APPENDIX B

The warpage problem has been referred to several times, and one cannot go far in welding without encountering this tendency. In the early days of the art, failure to foresee and understand this tendency was responsible for many failures and to unfair condemnation of the welding art. In some applications this tendency to warp or distort is so complex that it is difficult to analyze. But a consideration of the physical laws involved when a portion of two pieces of metal is raised in temperature

to fusion locally is helpful to the beginner, and on simple problems will either enable him to foresee the tendency and avoid it or at least to analyze the cause when trouble does develop.

Practically every butt weld takes place under the following conditions: Two pieces of metal are placed in juxtaposition, either with clearance or in contact. It is scarcely practicable to clamp these parts so rigidly that no relative movement is possible. Often it is undesirable. When the adjacent surfaces are raised in temperature to fusion by the arc, and the molten electrode is deposited so as to bond the two bodies into one, there is local expansion of the heated portions. The strains set up in each piece owing to the heat gradient are complex and impossible to calculate. Consider Fig. 13, A, and assume a molten zone owing to the immediate play of the arc. This will be surrounded by a zone of metal heated to a forgeable or plastic state. Its boundary will be the dull cherry red. Beyond this the metal becomes rigid. In this outer rigid zone are the greatest stresses (owing to the varying expansion of the metal). There is, of course, no sharply defined line of demarkation between the rigid and the plastic zones. Even where the metal looks black to the eye, improved malleability may be observed.

Just where these stresses exceed the elastic limit is hard to determine, but it is obvious that the metal in the plastic zone will flow freely to adjust itself to the expansion. If the two pieces are in contact and tightly clamped, they will upset where contact occurs, because the expansion due to the heat will cause the pieces to press against each other. This mutual pressure prevents freedom of expansion in the plane of the plates, and as the line of least resistance is transverse to this plane, the expansion manifests itself in a thickening of the plates. However, stress owing to this action is absorbed by the plasticity produced during welding. In the outer zone where the metal shows no color, there will be an area where the heat gradient is so low as to cause expansion within the elastic limit, and therefore upon cooling, this metal tends to resume its original dimensions. Adjoining this and toward the center will be a zone where the heat has caused a reduced elastic limit and increased the stress to a degree that the elastic limit is exceeded, causing permanent strain or distortion. Within this zone is one where the heat at its maximum has caused plasticity, with complete freedom of adjustment to the surrounding metal.

After the arc has passed, this metal cools and shrinks freely until the forging temperature is passed. Obviously the shrinkage must be at the expense of the thickness of the plate. After the central zone has cooled below forgeability, further cooling results in permanent stress in this metal and that surrounding it. Also, the metal in the first zone where the elastic limit has not been exceeded will be under strain because it adjoins the metal permanently cold-worked.

As the arc of the welder travels along the seam, these surrounding zones of plastic cold-forging and mildly stressed metal accompany it, and, as they cool, the trail becomes a set of parallel belts under varying degrees of stress. The general tendency owing to the final shrinkage is to draw the plates together; hence to bend them toward the weld. In Fig. 13, *B*, the two sheets being welded may be compared to a set of shears having the fulcrum at *a*, where the weld has just become rigid. At *b*, where it is changing from plastic to rigid, with accompanying shrinkage, is the power tending to close the shears. And this drawing together will be greatly magnified at *c*.

Another principle is of vital importance to the designer. Welds as now produced commercially are much less ductile than the mild steel on which they are generally made. Therefore, to secure the strength generally needed, the parts should be so disposed as to eliminate any possibility of concentrated stresses resulting from the outside applied forces. Designs can generally be made so that all of the load is of the same character (shear, tension, or compression) and uniform throughout the weld. This point can best be shown by illustration. Consider the stress in the weld in Fig. 18, *A*. The load *W* has a leverage of *a*. The weld is clearly at a disadvantage because the resisting moment is within the limits of the weld itself. In the cross-section through *m-n*, we readily recognize compression at *m* and tension at *n*. These forces become extreme at the edges of the section *m-n*. If the weld were ductile, a force at *W* would cause cold-drawing and upsetting of the metal of the weld. But as they are usually made, a slight force at *W* will readily crack the weld as though it were cast iron.

In *B* we have a practical design to carry load. The resisting capacity of the minimum area of the two welds is through the planes *m-n*. Through these planes, therefore, the greatest stress will occur. The measure of the capacity of the joint is the

resistance to shear of this area multiplied by its lever arm b and divided by a , the arm of the load.

However, this is not the most efficient weld for the case, as the stress is still somewhat concentrated at c . If loaded to failure, it would probably be from a progressive rupture starting at c (on the assumption of a rigid beam).

If the weld is applied as shown in C , we will have the condition specified at the beginning; namely, a load all of the same character (tension) and uniformly distributed. There is no concentration of load on this weld, and if it fails, it will be from a load W sufficient to rupture the entire section through $m-n$. The designer should try so to dispose the members that welds of this character can be applied. A simple test to apply to any weld for soundness of design is, Does the center of moments come inside or outside of the weld? If inside, the weld is at a disadvantage; if outside, the design is correct, and the safe load will be a simple function of the cross-section of the weld times its distance from the center. To be strictly accurate, resisting moment of the weld should be determined by integration, especially if the weld approaches closely to the fulcrum k . However, the designer will generally be able to estimate the strength of the joint as closely as the welder will make it, on the foregoing assumption.

In A the center of moments of W , the applied load, and of R , the resistance of the weld, is within the mass of the weld. This, as noted, will give rise to tension on one side and compression on the other. Hence the design is faulty. In both B and C the center of moments is at the corner k at considerable distance from the weld body, hence the design is practical in each case. In B notice that extra weld metal deposited at d brings the center of moments within the weld again and is useless (as we assume no flexibility of the beam). If extra metal is to be deposited, it should reinforce the original weld, as at e . In other words, the more the mass of the weld can be concentrated at the greatest possible distance from the fulcrum or center of moments, the more effective is its distribution, because this condition results in minimum variation in fiber stress in the weld. For this reason any variation of B is less effective than C , where the mass of the weld is concentrated on a line transverse to the plane of moments.

When these two principles are understood, the welder has the mechanics of the art, and if he is a mechanic he can adapt his set-up or design so as to eliminate or nullify this evil combination. If he is not naturally mechanical, he will never make a first-class welder. All propaganda to the contrary notwithstanding, the more brains a man has the better welder he will make. Also, some knowledge of metallurgy is highly desirable.

APPENDIX C

Before the first experimental cell was scrapped, permission was secured for a destruction test. It was picked up by a shop

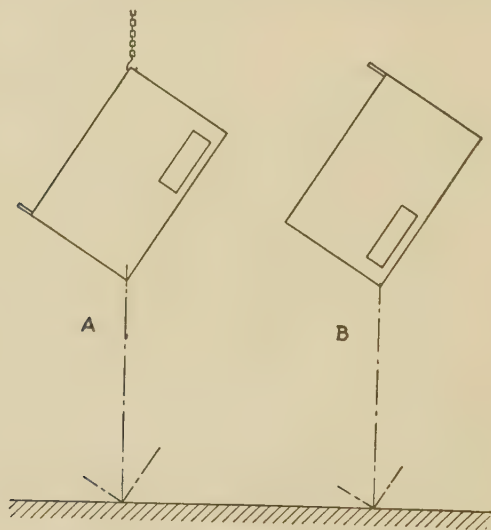


FIG. 16.—Cell suspended for drop test.

crane and dropped by means of a trigger release from various heights. While suspended, as shown in Fig. 16, it was first dropped 9 in. A slight buckle (Fig. 7) developed at the point marked *a*. It is believed that the cell could have been set up and used after this first drop. The next drop was from a height of $1\frac{1}{2}$ ft., resulting in a slight increase in the distortion at *a*, but no welds were ruptured. The next drop was from a height of $5\frac{1}{2}$ ft. The buckling of the roof became more pronounced and also began to develop on the opposite side. The floor plate also began to buckle adjoining the point of impact (Fig. 7, *b*). There were three blows on the base. We were allowed one more

test, and it was determined to make this effective, and so the cell was so suspended that it would strike on the upper rear edge. The previous blows were received on the bottom, and were probably borne largely by the heavy base angles. The upper rear edge had no backing, except a flange in the $\frac{1}{8}$ -in. sheet.

The last drop was from a height of 20 ft. Figure 7, *c* shows the results after the final drop. It will be noted that the bottom edge, marked *b*, is still in good condition, but that the top edge *c* was effectively crushed by the final blow. The displaced pieces showing in the opening are asbestos partitions. In general, the structure showed itself amply strong to stand the most violent handling conceivable in transportation.

APPENDIX D

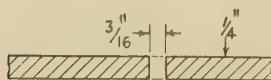
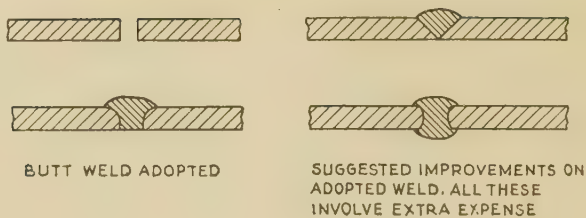
Considerable apprehension was expressed when it was proposed to weld butt joints without beveling. It was argued that such welds would lack penetration and that the deposited metal would be eccentrically placed relative to the axis of the bar. We were urged either to bevel our joints or to weld on both sides. Our butt weld is represented by the section shown in Fig. 17.

The proposed additional weld is shown adjoining in this sketch. Several test bars of butt welds having varying gaps were "pulled" with the following results:

APPROXIMATE AVERAGE OF THREE PIECES OF $\frac{1}{4}$ -IN. STOCK

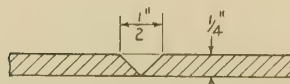
	Elastic limit, pounds	Yield point, pounds	Ultimate, pounds	Position of break
Unwelded bar.	23,350	33,000	52,000	
Butt with $\frac{1}{16}$ -in. gap. .	24,000	33,600	37,600	All at weld
Butt with $\frac{1}{8}$ -in. gap. . .	25,500	34,200	42,500	All at weld
Butt with $\frac{3}{16}$ -in. gap. . .	27,000	34,500	50,400	One at weld; two outside of weld
Butt with $\frac{1}{4}$ -in. gap. . .	24,600	33,870	50,400	One at weld; two outside of weld

NOTE.—All of the $\frac{1}{16}$ - and $\frac{1}{8}$ -in. gap specimens broke at weld; two of the three $\frac{3}{16}$ - and $\frac{1}{4}$ -in. gap pieces broke outside of the weld.



VOID TO BE FILLED IN WELD
AS ADOPTED

$$\frac{3}{16} \times \frac{1}{4} = \frac{3}{64} = 0.047 \text{ SQ. IN.}$$



VOID TO BE FILLED IN SUGGESTED V-WELD

$$\frac{1}{2} \times \frac{1}{4} \times \frac{1}{2} = \frac{1}{16} = 0.063 \text{ SQ. IN.}$$

FIG. 17.—Section showing butt weld.

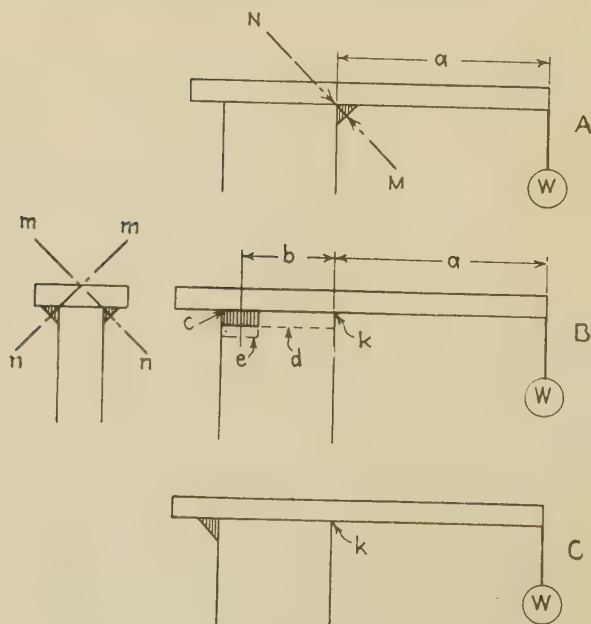


FIG. 18.—Stresses affecting the weld.

From the foregoing results, it seems clear that with the adoption of a $\frac{3}{16}$ -in. gap for butt welds in $\frac{1}{4}$ -in. stock, highly satisfactory welds may be obtained with small deposition of weld metal on only one side. Thus no loss of time is experienced in turning over and reclamping the pieces or in laying on the additional weld. The area of cross-section of void with our $\frac{3}{16}$ by $\frac{1}{4}$ -in. butt is $\frac{3}{64}$ sq. in. = 0.047 sq. in., while the void to be filled with a $\frac{1}{4}$ -in., 45° beveled butt is one-half of $\frac{1}{4}$ by $\frac{1}{2}$ (see Fig. 18 at *b*), or $\frac{1}{16}$ sq. in. = 0.062 sq. in., or one-half greater. Thus with the beveled butt extra time is required to do the beveling and more weld must be deposited. There should be no generalization as to this $\frac{3}{16}$ by $\frac{1}{4}$ ratio being effective with thicker plates. No study has been made with any but $\frac{1}{4}$ -in. material.

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